ERDC/CHL TR-01-10

Coastal and Hydraulics Laboratory

US Army Corps of Engineers®
Engineer Research and Development Center

Design for Small-Boat Harbor Improvements and Tidal Flushing, St. Paul Harbor, St. Paul Island, Alaska

Coastal Model Investigation

Robert R. Bottin, Jr., and Hugh F. Acuff

May 2001

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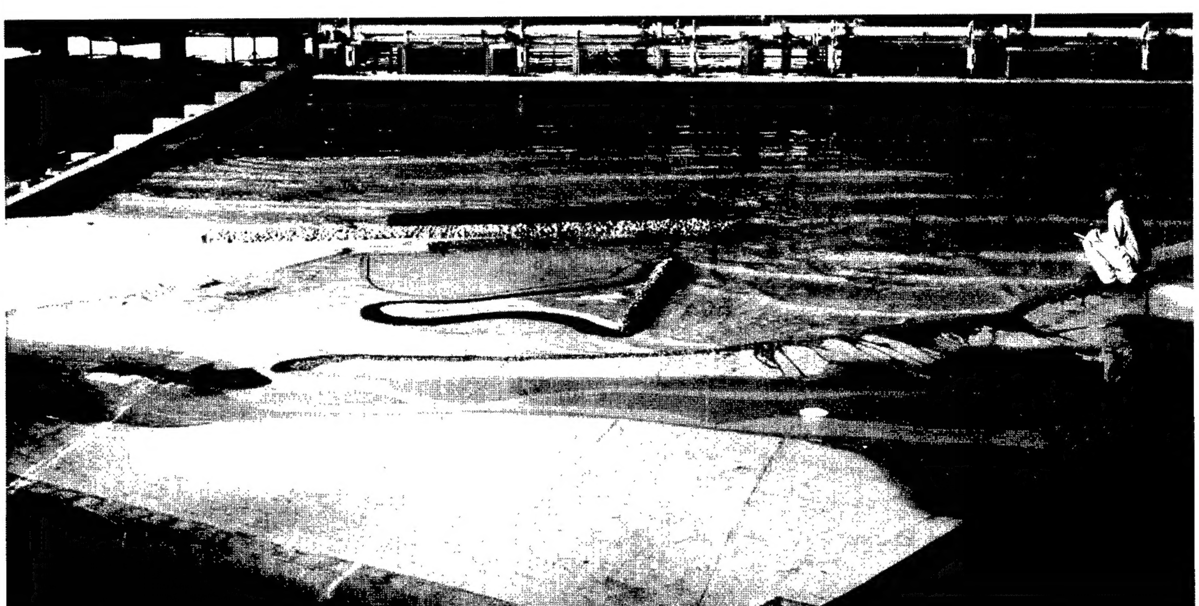

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Design for Small-Boat Harbor Improvements and Tidal Flushing, St. Paul Harbor, St. Paul Island, Alaska

Coastal Model Investigation

by Robert R. Bottin, Jr., Hugh F. Acuff

Coastal and Hydraulics Laboratory
U.S. Army Engineer Research and Development Center
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Final report

Approved for public release; distribution is unlimited

Prepared for U.S. Army Engineer District, Alaska
Anchorage, AK 99506-0898

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Preface

A request for a model investigation to study harbor modifications at St. Paul Harbor, St. Paul Island, AK, was initiated by the U.S. Army Engineer District, Alaska, in a letter to the U.S. Army Engineer Division, Pacific Ocean. Authorization for the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL), to perform the study was subsequently granted by Headquarters, U.S. Army Corps of Engineers (HQUSACE). Funds were provided by the Alaska District in December 2000.

Model experiments were conducted at ERDC during January 2001 by personnel of the Harbors and Entrances Branch (HEB), CHL, under the direction of Messrs. Thomas W. Richardson and Thomas J. Pokrefke, Jr., Acting Director and Acting Assistant Director of CHL, respectively; and under direct supervision of Mr. Dennis G. Markle, Chief of HEB. Model experiments were conducted by Messrs. Hugh F. Acuff, Glenn B. Myrick, and Larry R. Tolliver, and Ms. Kristi L. Evans, civil engineering technicians, and Mr. William G. Henderson, computer assistant, under the supervision of Mr. Robert R. Bottin, Jr., research physical scientist. This report was prepared by Messrs. Bottin and Acuff.

Messrs. Ken Eisses and Alan Jeffries were technical points of contact for the Alaska District. The following personnel from the Alaska District visited ERDC to observe and participate in model operations during the study: Messrs. Mr. Ken Eisses, Mr. Alan Jeffries, Mr. John Burns, and Mr. John Oliver, consultant, for the Alaska District.

Initial results for the model were reported in Technical Report CERC-96-7, "Study of Harbor Improvements at St. Paul Harbor, St. Paul Island, Alaska," September 1996, and results for the initial reactivation of the study were reported in Miscellaneous Paper CHL-97-7, "Study for Flushing of Salt Lagoon and Small-Boat Harbor Improvements at St. Paul Harbor, St. Paul Island, Alaska," August 1997. Results for the reactivated model finalizing the design of small-boat harbor improvements and flushing at St. Paul Harbor are reported herein.

Dr. James R. Houston was Director of ERDC during model operation and the preparation and publication of this report, and Mr. Armando J. Roberto, Jr., was Acting Commander.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acres	4,046.873	square meters
cubic feet per second	0.02831685	cubic meters per second
cubic yards	0.7646	cubic meters
degrees (angle)	0.01745329	radians
feet	0.3048	meters
feet per second	0.3048	meters per second
inches	2.54	centimeters
knots (international)	1.8532	kilometers per hour
miles (U.S. statute)	1.609347	kilometers
miles per hour	1.609347	kilometers per hour
pounds (mass)	0.4536	kilograms
pounds (mass) per cubic feet	16.02	kilograms per cubic meter
square feet	0.09290304	square meters
square miles (U.S. statute)	2.589988	square kilometers
tons (2,000 lb, mass)	907.1848	kilograms

1 Introduction

Prototype

St. Paul Island is the northernmost and largest island of the Pribilofs in the eastern Bering Sea (Figure 1) with a land area of 114 sq km (44 sq mi).[1] The Pribilofs are of volcanic origin, and St. Paul Island is composed predominantly of volcanic materials in the form of lava flows and loose cinders with sandy deposits. The west and southwest portions of the island are high and mountainous with precipitous cliffs along the coast. The remainder of the island is low and rolling with a number of extinct volcanic peaks scattered throughout. Only two of the Pribilof Islands are populated, St. Paul with about 800 people and St. George with approximately 300 residents. Two-thirds of the St. Paul population is Alaska native.

The Pribilof Islands support large populations of birds, mammals, fish, and invertebrates. The Pribilofs are the primary breeding ground for northern fur seals where approximately two-thirds of the world's population (1.3 to 1.4 million) migrate annually (U.S. Army Engineer District, Alaska, 1981). More than a quarter million sea birds nest on St. Paul Island each year, mainly along the coastal cliffs. The uplands are inhabited by songbirds, white and blue foxes, and a transplanted herd of approximately 250 reindeer. The island is treeless and covered with grasses, sedges, and wildflowers. The eastern Bering Sea near St. Paul supports populations of shrimp, commercially harvestable species of crab, and bottom fish.

The city of St. Paul is located on a cove on the southern tip of the island and is the island's only settlement. The islands were originally settled by the Russians to harvest fur seals. The treaty for the purchase of Alaska from Russia by the United States in 1867 placed the Pribilofs under United States control. The National Marine Fisheries Service (NMFS) and its predecessor Federal agencies were responsible for the fur seal industry in the Pribilofs since 1911, managing the harvest according to a series of international agreements between the United States, Canada, Japan, and the Soviet Union. In 1983, the harvest of fur seals was discontinued due to a seal harvest moratorium. The NMFS

[1] Units of measurement in this report are shown in SI (metric) units, followed by non-SI (British) units in parentheses. In addition, a table of factors for converting non-SI units of measurement used in figures, plates, and tables in this report to SI units is presented on page vi.

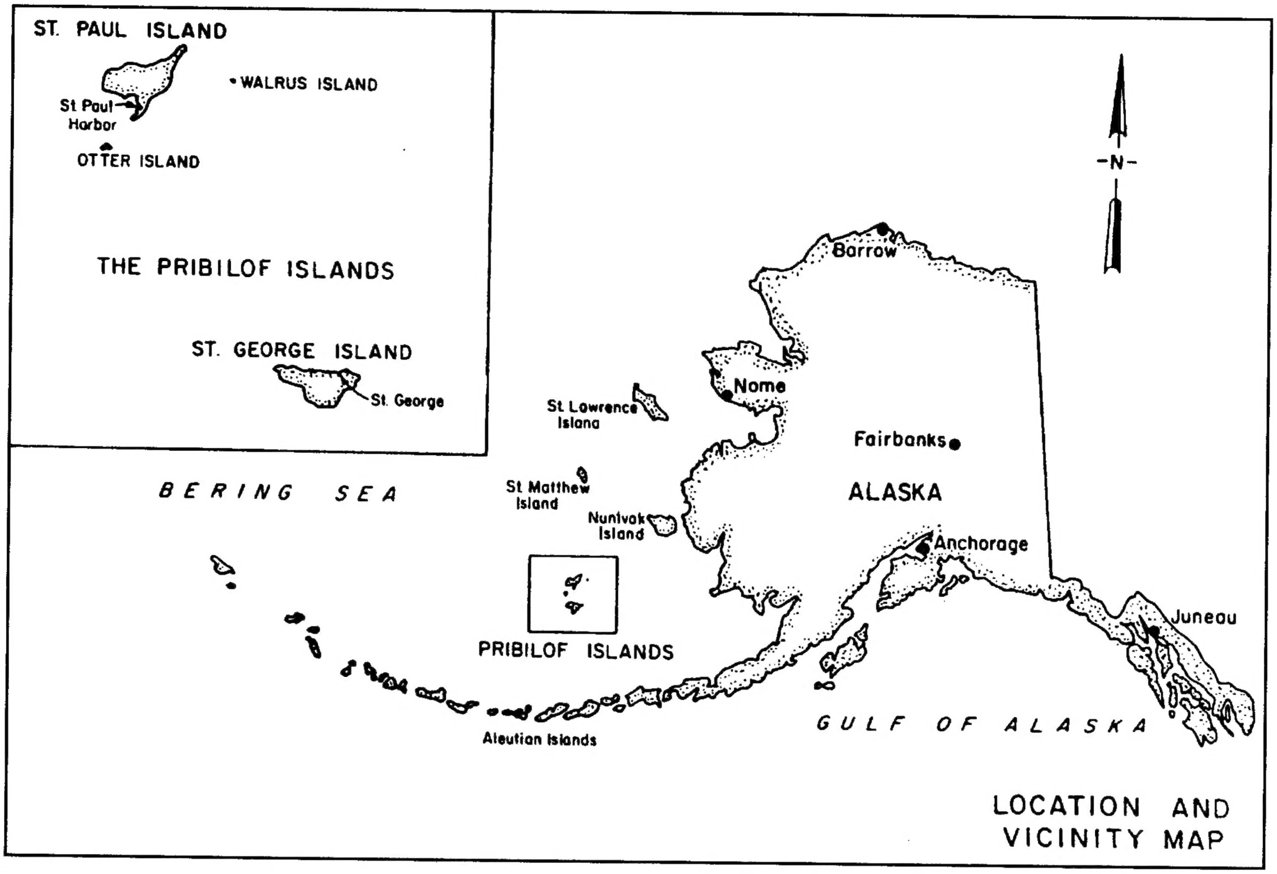

Figure 1. Project location

terminated administration, management, and employment at St. Paul. This event had a significant adverse impact on the economy, and the standard of living could not be maintained. At that time the village had no other economic base, no harbor infrastructure, inadequate and unpermitted utilities, overcrowded housing, high unemployment, and limited air and vessel transportation. Development of a harbor and associated marine-related industries fulfilled the need for new sources of employment and income on the island.

Harbor Development

A breakwater was constructed at St. Paul in Village Cove in 1983, but subsequently failed during storms of 1984. A new breakwater was designed and constructed by Tetra Tech, Inc., consultants to the city of St. Paul (Tetra Tech, Inc. 1987). The structure was 229 m (750 ft) in length and functioned well, in regard to stability, during the 1985 and 1986 winter seasons. A 61-m-long (200-ft-long), vertical-wall dock was installed in the lee of the breakwater in 1986 to accommodate fishing vessels. The breakwater, however, was not of sufficient length to provide wave protection to vessels using the dock, particularly during storm events.

In 1989, construction of the current harbor configuration was completed. A layout of the harbor is shown in Figure 2. It consisted of a 549-m-long

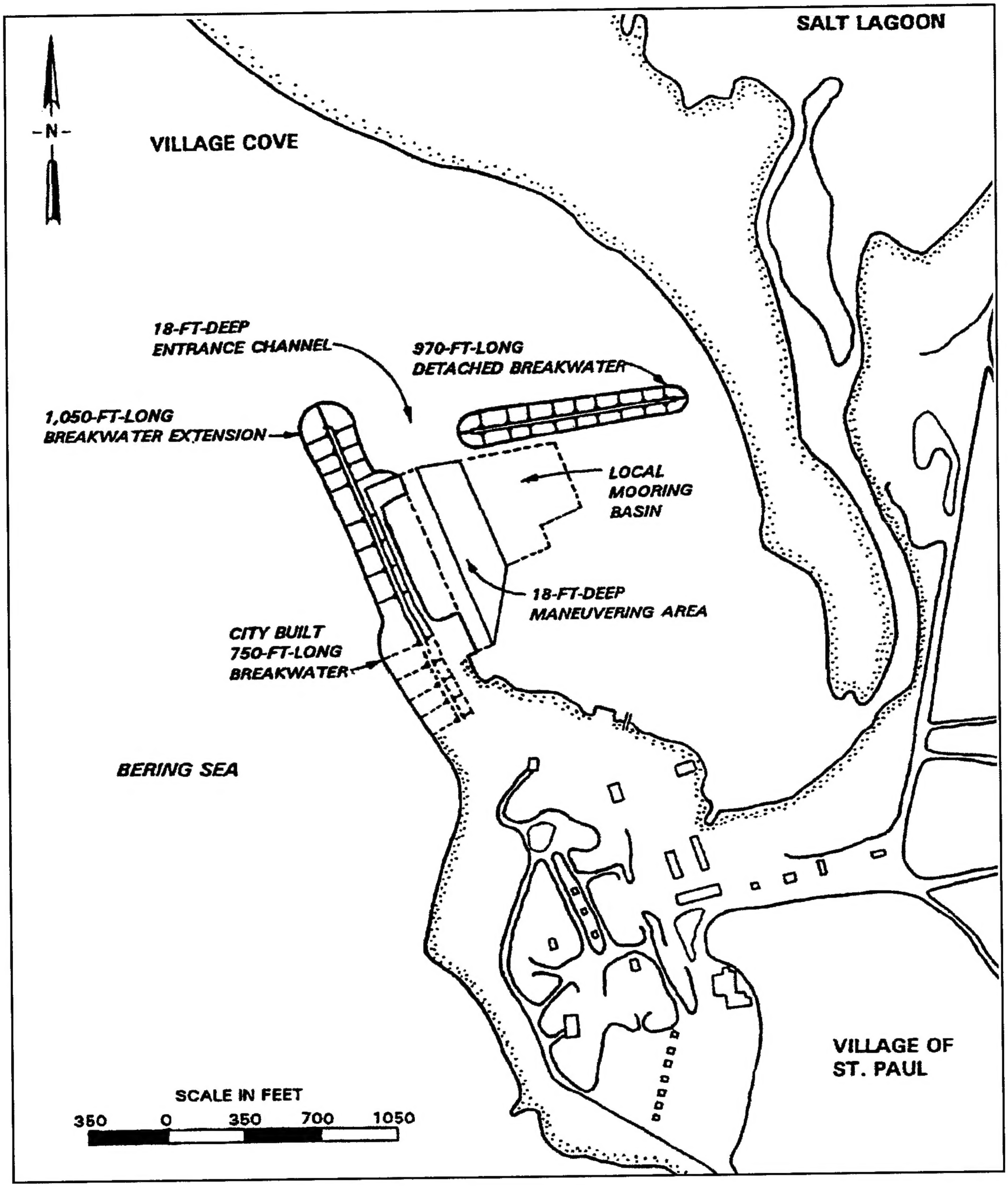

Figure 2. Layout of St. Paul Harbor (Scale is in feet to convert feet to meters, multiply by 0.3048)

(1,800-ft-long) main breakwater, a 296-m-long (970-ft-long) detached breakwater, and space for 274 m (900 ft) of docks on the lee side of the main breakwater. The main breakwater generally follows the -7.6-m (-25-ft)[1] contour in Village Cove and results in a harbor with 32,375 to 40,470 sq m (8 to 10 acres) of area and water depths of 5.5 to 7.6 m (18 to 25 ft) on the lee side of the breakwater. The center line of the detached breakwater makes an interior angle of 75 deg with the main structure at sta 17+00, and provides a 91-m-wide (300-ft-wide) harbor entrance. A 61-m-wide (200-ft-wide) opening between the eastern end of the detached breakwater and the shore is maintained to enhance harbor circulation. An aerial photograph of the existing St. Paul Harbor is shown in Figure 3.

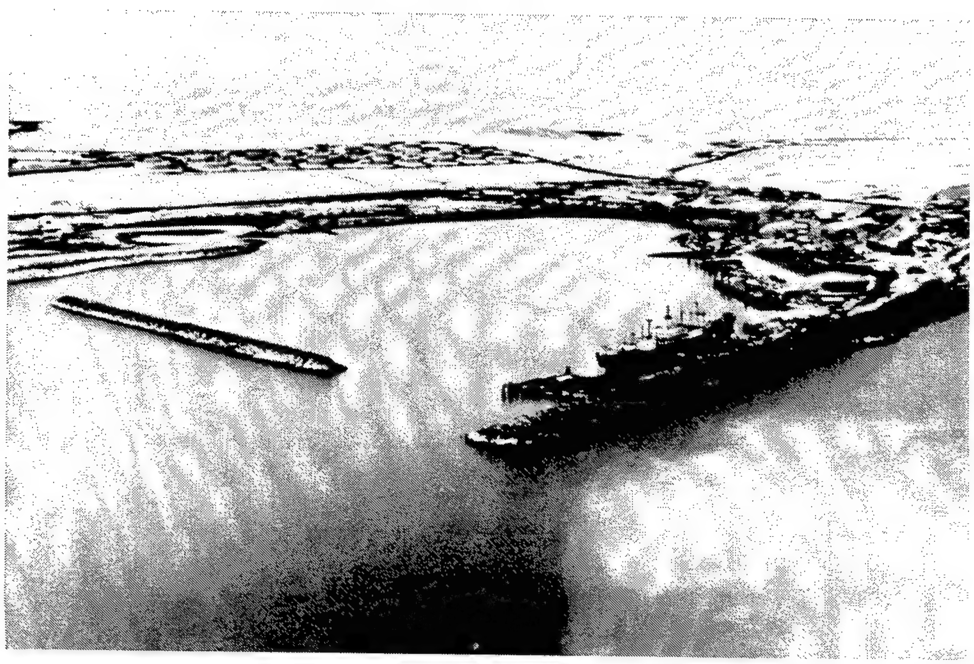

Figure 3. Aerial view of St. Paul Harbor

The main breakwater has a design crest elevation (el) of +11.3 m (+37 ft) from sta 7+50 to a point approximately 15.2 m (50 ft) north of the northernmost dock. The remaining portion of the structure has a crest el of +9.1 m (+30 ft). Armor stone used on the breakwater trunk was 16,330-kg (18-tons), and 21,770-kg (24-ton) armor stone was used on the head. The slope of the trunk is 1V:2H with a 1V:2.5H slope around the breakwater head. Special placement of the armor stone was specified in the contract documents which required orientation of the long axis of each stone normal to the breakwater slope. A roadway was constructed on the lee side of the main breakwater adjacent to the proposed docks. The detached breakwater has a crest el of +5.5 m (+18 ft) with 4,535-kg (5-ton) armor stone placed on a slope of 1V:1.5H. Prior to construction of the 1989 improvements, both two-dimensional (Ward 1988) and three-dimensional (Bottin and Mize 1988) hydraulic model investigations were

[1] All contours and elevations cited herein are in meters (feet) referred to mean lower low water (mllw) unless otherwise noted.

conducted at the U.S. Army Engineer Research and Development Center (ERDC) to optimize structural and functional design of the harbor.

After its construction in 1989, the harbor experienced a rapid growth cycle and quickly became overcrowded. In the mid 1990s, St. Paul Harbor served a fleet of 230 transient vessels during the crabbing season. A total of 27 floating processors were located within a 4.8-km (3-mile) limit of the harbor. In addition, three processing plants had permanently located within the harbor complex (U.S. Army Engineer District, Alaska, 1995). Subsequent to harbor construction, significant overtopping of the main breakwater had been experienced during the winter seasons. Overtopping may have been caused by larger than design storm waves and/or still-water levels or possible settlement and consolidation of the breakwater stone. Overtopping caused the roadway in the lee of the breakwater to wash out, and repairs were required frequently during the storm season. Due to these problems and needs, the harbor was again studied at ERDC in 1996. The feasibility of deepening the entrance channel and dredging a deeper and larger maneuvering basin was proposed to relieve the congestion in the harbor. In addition, a submerged reef breakwater concept was studied as a means of reducing wave overtopping of and wave transmission through the main breakwater. Two- and three-dimensional model investigations were conducted by Ward[1] and Bottin (1996), respectively, to optimize reef breakwater cross sections and layout as well as wave and current conditions in the harbor.

Construction of three parallel, submerged reef breakwaters seaward of the main breakwater was initiated during the 2000 construction season. The reef structures were constructed with 455- to 3,630-kg (1,000- to 8,000-lb) stone at an el of -3.7 m (-12 ft) with side slopes of 1V:1.5H. They were 380 m (1,250 ft) in length. The shoreward crest of the innermost reef was 52 m (170 ft) from the toe of the existing main breakwater. The crest widths of the reefs were 6.1 m (20 ft), and the crests were 21.3 m (70 ft) apart. The reefs were placed on bedding stone that ranged from 9 to 225 kg (20 to 500 lb). A layout of the reef breakwaters is shown in Figure 4. In addition, the contract included that a total of 25 selected armor stones be placed in voids that had occurred in the main breakwater due to armor displacement during storms. In June 2000 these armor stones were placed on the breakwater along the waterline between stations 8+80 and 9+70. Also included in the contract was the placement of 75 selected armor stones in damaged areas of the detached breakwater at St. Paul Harbor. Offshore reef construction was only partially completed during the 2000 season, and final construction will be completed during the summer of 2001.

Previously Reported Model Experiments and Conclusions

The 1:100-scale St. Paul Harbor model was initially constructed to investigate the feasibility of deepening the entrance channel and dredging a

[1] Ward, D. L. (1996). "Runup and overtopping studies for St. Paul Harbor Breakwater, St. Paul, Alaska," (unpublished), U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

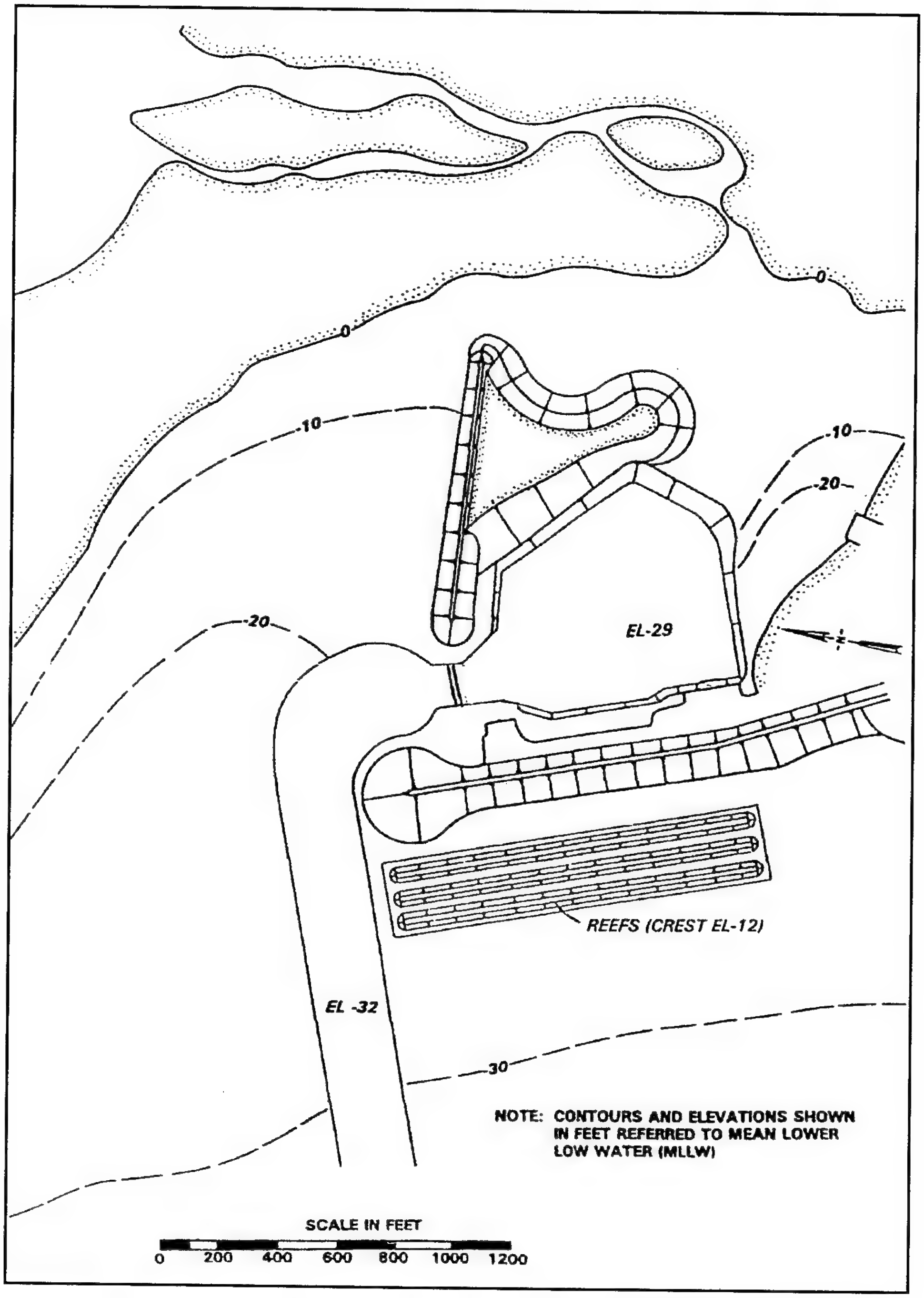

Figure 4. Layout of submerged reef breakwaters (Scale is in feet, to convert feet to meters, multiply by 0.3048)

deeper and larger maneuvering basin to relieve the current congestion. The impacts of proposed harbor improvements on wave conditions, wave-induced current patterns and magnitudes, and sediment patterns and subsequent deposits in the harbor were studied. In addition, the impacts of a proposed submerged reef breakwater were investigated relative to wave-induced current patterns and magnitudes and sediment tracer patterns and subsequent deposits seaward of the main breakwater. Details of the investigation were published (Bottin 1996), and

conclusions derived from results of those experiments are shown as follows. Plan numbers refer to those in the 1996 investigation.

a. During periods of severe storm wave activity with high tide conditions, wave heights in the existing harbor will exceed 1.7 m (5.5 ft) along the dock in the lee of the main breakwater and 0.8 m (2.5 ft) at the Tanadgusix (TDX) dock.

b. For existing conditions, currents enter the harbor through the opening at the shoreward end of the detached breakwater and move in a clockwise direction exiting through the entrance. Maximum velocities along the shoreline inside the harbor will exceed 2.5 m/s (8 fps). Currents also move seaward along the sea side of the detached breakwater across the harbor entrance.

c. For existing conditions, sediment moves southerly along the boulder spit and enters the harbor through the opening at the shoreward end of the detached breakwater. Sediment also moves westerly along the sea side of the detached breakwater toward the harbor entrance.

d. Experimental results obtained for the initial submerged reefs (Plans 1 and 2) indicated the structures would have no adverse impact on current patterns and magnitudes or sediment tracer patterns and deposits seaward of the main breakwater.

e. An extension of the initial submerged reefs northerly by 122 m (400 ft) in length (Plan 4) will decrease wave heights in the approach and entrance channels and result in improved navigation conditions.

f. A 15.2-m (50-ft) reduction in the length of the submerged reefs (from 396 to 381 m (1,300 to 1,250 ft)) on their southern end (Plan 9) will not increase wave conditions in the harbor.

g. Experimental results for the deepened channel and maneuvering area and the 381-m-long (1,250-ft-long) submerged reefs of Plan 10 indicated that wave heights would increase at the TDX dock and the inner harbor area when compared to existing conditions.

h. Installation of the wave-dissipating spending beach in the harbor (Plan 11) with the deepened channel and maneuvering area and the 381-m-long (1,250-ft-long) submerged reefs will result in reduced wave conditions. Wave heights throughout the harbor will be significantly less than those obtained for existing conditions.

i. Installation of Plan 10 (deepened channel and maneuvering area and the 381-m-long (1,250-ft-long) submerged reefs) or Plan 11 (addition of the wave-dissipating spending beach) will have no adverse impact on current patterns and magnitudes and/or sediment patterns and subsequent deposits in the vicinity of the harbor.

j. The 120-m-long (400-ft-long) breakwater spur of Plan 12 will have no adverse impact on wave or current conditions in the harbor. It will,

however, redirect sediment movement and subsequent deposits from the entrance channel to the northerly edge of the channel, and thus, reduce the potential for shoaling.

The 1:100-scale St. Paul Harbor model was reactivated to determine the impacts of proposed small-boat harbor modifications on wave conditions, current patterns and magnitudes, and sediment movement patterns and subsequent deposits within the complex. In addition, experiments were conducted to study both wave-induced and tidal flushing of Salt Lagoon. Details of this investigation were published (Bottin and Acuff 1997), and conclusions derived from results of those experiments are shown as follows. Plan numbering began where they ended in the 1996 study.

a. Preliminary experiments (Plans 13-18) revealed that all improvement plans would result in wave heights of less than 0.3 m (1.0 ft) in the small-boat mooring areas.

b. Preliminary experiments indicated that with the originally proposed plans, sediment deposits would occur in the small-boat navigation channel. A breakwater extending southeasterly from the wave-dissipating spending beach, or an extension of the spending beach, however, would prevent shoaling of the channel.

c. Preliminary experiments revealed that the location of the north breakwater was critical with respect to diverting tidal currents from the lagoon connecting channel toward the harbor basin and providing circulation.

d. Of the improvement plans investigated with the wave energy channel connected to Salt Lagoon north of the harbor, the 61-m-wide (200-ft-wide), +0.9-m (+3.0-ft) el channel of Plan 21 was optimum with respect to those configurations.

e. The improvement plan configurations of Plans 24 and 25 (26-vessel and 52-vessel basins, respectively) will provide adequate wave protection, shoaling protection, and harbor circulation for the new small-boat harbor.

f. Improvements in shoaling and circulation conditions for the existing harbor will be obtained with the installation of the sediment deposition basin, the southeasterly extension of the wave-dissipating spending beach, and the north breakwater (Plan 26).

Purpose of the Current Investigation

At the request of the Alaska District, the 1:100-scale hydraulic model of St. Paul Harbor was reactivated by ERDC to finalize the design of proposed small-boat harbor modifications based on wave conditions and current patterns and magnitudes within the complex. Long-period seiche conditions within the harbor were also evaluated. In addition, experiments were conducted to study tidal flushing of the harbor from Salt Lagoon. An expedited testing program was performed with a minimum number of experimental conditions.

2 Model

Design of Model

The St. Paul Harbor model (Figure 5) was initially constructed in 1996 to an undistorted linear scale of 1:100, model to prototype. Scale selection was based on the following factors:

a. Depth of water required in the model to prevent excessive bottom friction.

b. Absolute size of model waves.

c. Available shelter dimensions and area required for model construction.

d. Efficiency of model operation.

e. Available wave-generating and wave-measuring equipment.

f. Model construction costs.

A geometrically undistorted model was necessary to ensure accurate reproduction of wave and current patterns. Following selection of the linear scale, the model was designed and operated in accordance with Froude's model law (Stevens et al. 1942). The scale relations used for design and operation of the model were as follows:

Characteristic	Model-Prototype Dimension[1]	Scale Relations
Length	L	L_4 = 1:100
Area	L^2	$A_r = L_r^2$ = 1:10,000
Volume	L^3	$V_r = L_r^3$ 1:1,000,000
Time	T	$T_r = L_r^{1/2}$ = 1:10
Velocity	L/T	$V_r = L_r^{1/2}$ = 1:10
[1] Dimensions are in terms of length (L) and time (T).		

The existing breakwaters at St. Paul Harbor are rubble-mound structures. Experience and experimental research have shown that considerable wave energy passes through the interstices of this type structure; thus, the transmission and absorption of wave energy became a matter of concern in the design of 1:100-scale model. In small-scale hydraulic models, rubble-mound structures reflect

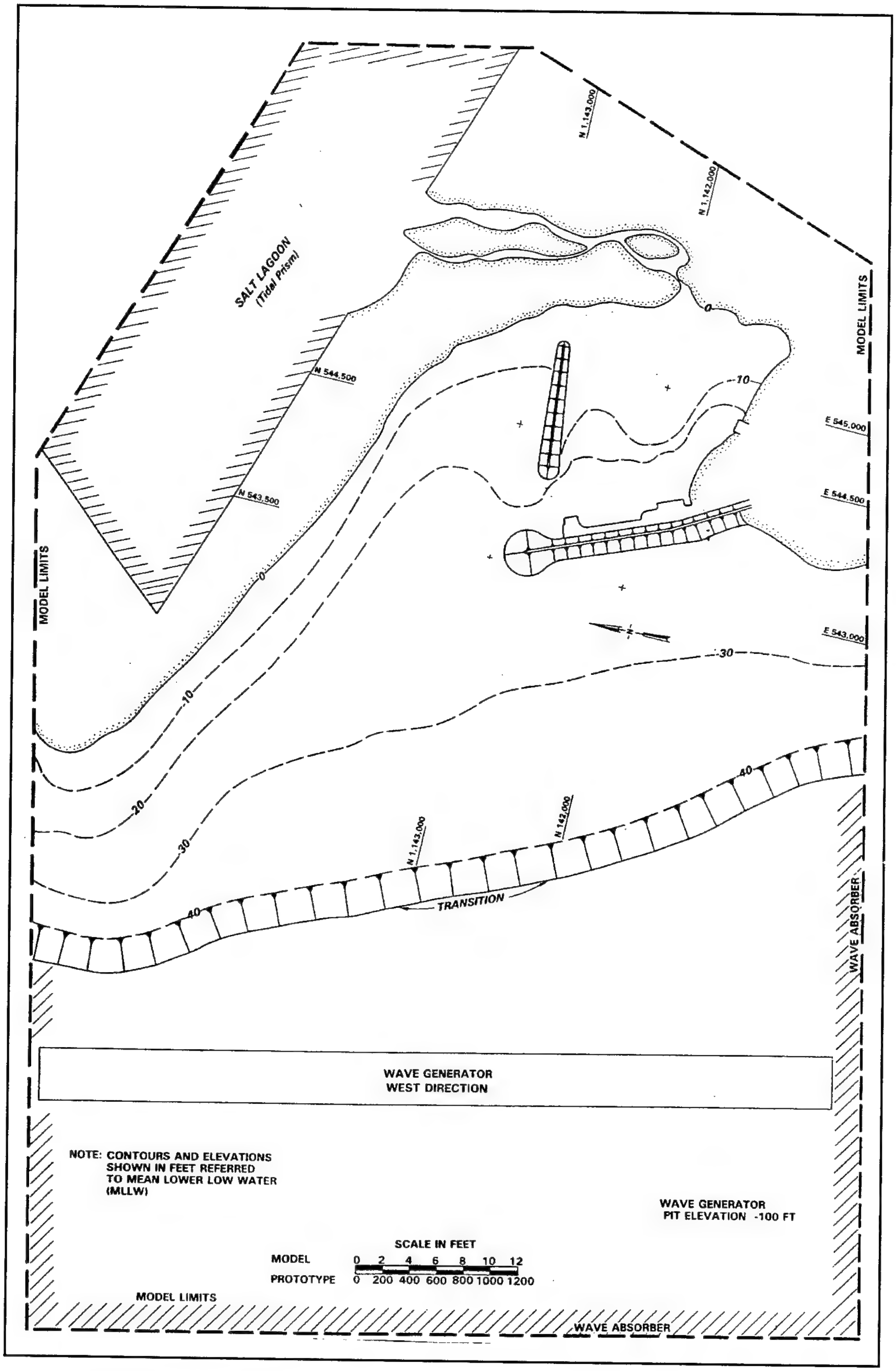

Figure 5. Model layout (Scale is in feet, to convert feet to meters, multiply by 0.3048)

more and absorb or dissipate less wave energy than geometrically similar prototype structures (LeMehaute 1965). Also, the transmission of wave energy through a rubble-mound structure is less for the small-scale model than for the prototype. Consequently, some adjustment in small-scale model rubble-mound structures is needed to ensure satisfactory reproduction of wave-reflection and wave-transmission characteristics. In past investigations (Dai and Jackson 1966; Brasfeild and Ball 1967) at ERDC, this adjustment was made by determining wave-energy transmission characteristics of the proposed structure in a two-dimensional model using a scale large enough to ensure negligible scale effects. A cross section then was developed for the small-scale, three-dimensional model that would provide essentially the same relative transmission and reflection of wave energy. Therefore, from previous findings for structures and wave conditions similar to those at St. Paul Harbor, it was determined that a close approximation of the correct wave-energy transmission and reflection characteristics could be obtained by increasing the size of the rock used in the 1:100-scale model to approximately two times that required for geometric similarity. Accordingly, in constructing the rubble-mound structures in the St. Paul Harbor model, rock sizes were computed linearly by scale, then multiplied by 2 to determine the actual sizes to be used in the model.

Model and Appurtenances

The model reproduced approximately 2,865 m (9,400 ft) of the St. Paul Island shoreline (from Tolsti Point easterly and then southerly to a point south of the existing breakwater trunk), the existing harbor, and underwater topography in the Bering Sea to an offshore depth of 12.2 m (40 ft) with a sloping transition to the wave generation pit elevation of -30.5 m (-100 ft). A small connecting channel to a salt lagoon (located east of the harbor) also was included in the model as well as the tidal prism of the salt lagoon. The total area reproduced in the model was approximately 605 sq m (6,500 sq ft), representing about 6 sq km (2.3 sq mi) in the prototype. Vertical control for model construction was based on mean lower low water (mllw), and horizontal control was referenced to a local prototype grid system. A general view of the model is shown in Figure 6.

Model waves were reproduced by an 18.3-m-long (60-ft-long), electrohydraulic, unidirectional, spectral wave generator with a trapezoidal-shaped plunger. The vertical motion of the plunger was controlled by a computer-generated command signal, and movement of the plunger caused a displacement of water which generated required test waves.

An automated data acquisition and control system, designed and constructed at ERDC, was used to generate and transmit wave generator control signals, monitor wave generator feedback, and secure and analyze wave data at selected locations in the model. Through the use of a microvax computer, the electrical output of parallel-wire, capacitance-type wave gauges, which varied with the change in water-surface elevation with respect to time, were recorded on magnetic disks. These data then were analyzed to obtain the parametric wave data.

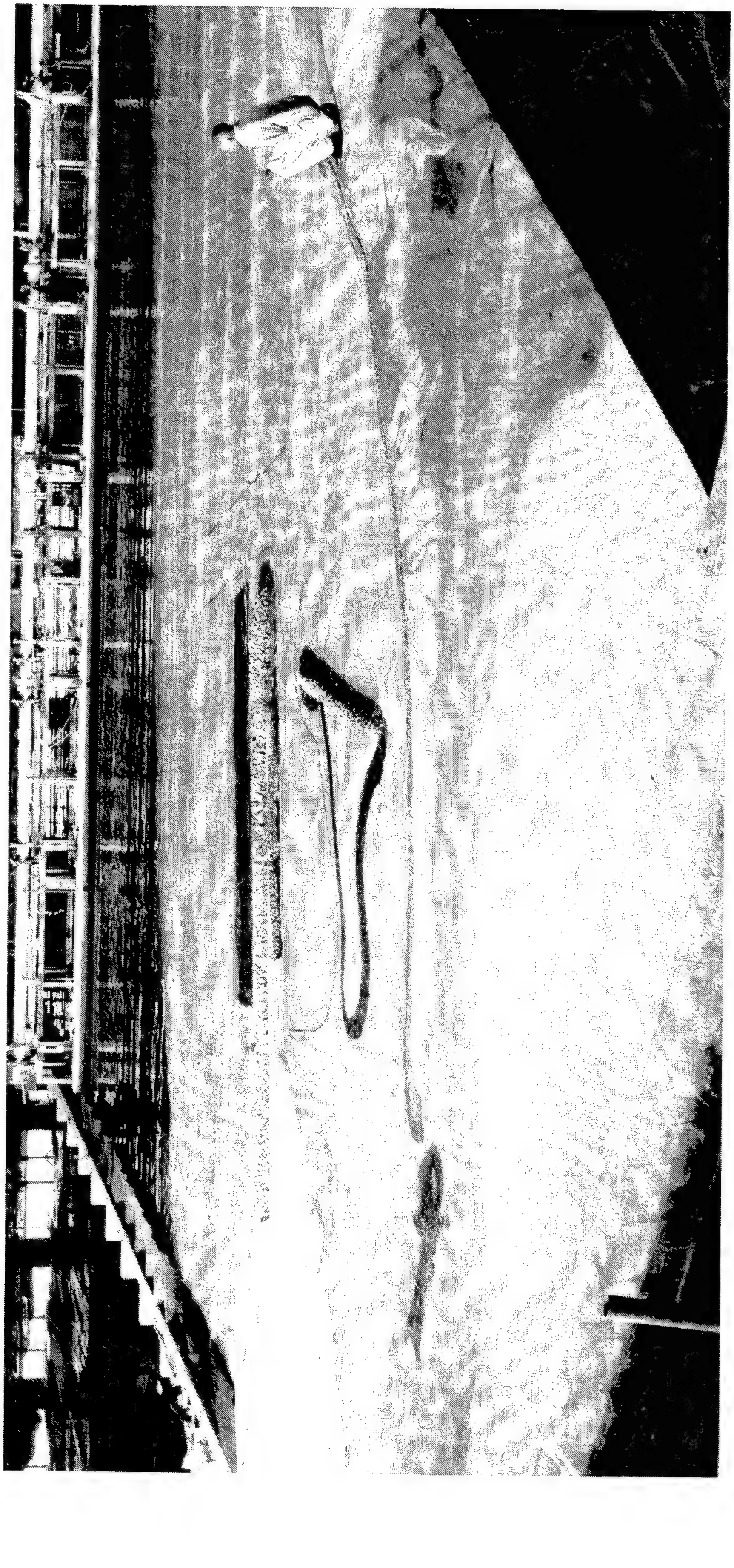

Figure 6. General view of model

A 0.6-m (2-ft) (horizontal) solid layer of fiber wave absorber was placed along the inside perimeter of the model to dampen wave energy that might otherwise be reflected from the model walls. In addition, guide vanes were placed along the wave generator sides in the flat pit area to ensure proper formation of the wave train incident to the model contours.

The St. Paul Harbor model facility did not include calibrated tidal reproduction facilities. These facilities require an enormous amount of time and funds to prepare. Since time and funds were limited, model tides were reproduced simply by raising (filling the basin) or lowering (draining the basin) the water level. The water level was raised and lowered linearly over the appropriate tidal period (36 min in the model which equates to 6 hr in the prototype).

3 Experimental Conditions and Procedures

Selection of Experimental Conditions

Still-water level

Still-water levels (swls) for wave action models are selected so that various wave-induced phenomena that are dependent on water depths are accurately reproduced in the model. These phenomena include refraction of waves in the project area, overtopping of harbor structures by waves, reflection of wave energy from various structures, and transmission of wave energy through porous structures.

In most cases, it is desirable to select a model swl that closely approximates the higher water stages which normally occur in the prototype for the following reasons:

a. The maximum amount of wave energy reaching a coastal area normally occurs during the higher water phase of the local tidal cycle.

b. Most storms moving onshore are characteristically accompanied by a higher water level due to wind, tide, and storm surge.

c. The selection of a high swl helps minimize model scale effects due to viscous bottom friction.

d. When a high swl is selected, a model investigation tends to yield more conservative results.

Swls of +1.0, +1.5, and +2.1 m (+3.2, +5.0, and +7.0 ft) were selected by the Alaska District for use during the initial experiments with the St. Paul model. Only the +1.0 and +2.1 m (+3.2 and +7.0 ft) swls, however, were used during the reactivated experimental series. The lower value (+1.0 m (+3.2 ft)) represents mean higher high water (mhhw). The higher value (+2.1 m (+7.0 ft)) was an extreme estimate based on observations made in the prototype during storm wave conditions.

Factors influencing selection of experimental wave characteristics

In planning the experimental program for a model investigation of harbor wave-action problems, it is necessary to select heights, periods, and directions for the test waves that will allow a realistic test of the proposed improvement plans and an accurate evaluation of the elements of the various proposals. Surface-wind waves are generated primarily by the interactions between tangential stresses of wind flowing over water, resonance between the water surface and atmospheric turbulence, and interactions between individual wave components. The height and period of the maximum significant wave that can be generated by a given storm depend on the wind speed, the length of time that wind of a given speed continues to blow, and the distance over water (fetch) which the wind blows. Selection of experimental wave conditions entails evaluation of such factors as the following:

a. Fetch and decay distances (the latter being the distance over which waves travel after leaving the generating area) for various directions from which waves can approach the problem area.

b. Frequency of occurrence and duration of storm winds from the different directions.

c. Alignment, size, and relative geographic position of the navigation structures.

d. Alignments, lengths, and locations of the various reflecting surfaces in the area.

e. Refraction of waves caused by differentials in depth in the area seaward of the site, which may create either a concentration or a diffusion of wave energy.

Wave refraction

When waves move into water of gradually decreasing depth, transformations take place in all wave characteristics except wave period (to the first order of approximation). The most important transformations with respect to selection of experimental wave characteristics are the changes in wave height and direction of travel due to the phenomenon referred to as wave refraction. During a previous model investigation (Bottin and Mize 1988), the change in wave height and direction at St. Paul Harbor was determined by using the numerical Regional Coastal Processes Wave Transformation Model (RCPWAVE) developed by Ebersole (1985). During the previous study, model experiments were conducted for five wave directions. For the current series, however, waves from only the west (259 deg) direction were used. The west direction was the most critical with respect to wave heights, wave-induced current patterns and magnitudes, and sediment tracer patterns at the harbor.

Prototype wave data and selection of experimental waves

Measured prototype data covering a sufficiently long duration from which to base a comprehensive statistical analysis of wave conditions were unavailable for the St. Paul Harbor area. However, in the previous model investigation (Bottin and Mize 1988), statistical deepwater wave hindcast data representative of this area were obtained from the Coastal Engineering Research Center (CERC), Wave Information Studies (WIS). Additional information on WIS may be obtained from Corson (1985). After a review of the data from the previous study, and due to limited time and funds for the current investigation, Alaska District selected the following waves for use in the current experimental series:

Period, sec	Height, m (ft)
8	3.0 (10)
10	3.0 (10)
16	4.4 (14.4)
	5.8 (19)
20	4.3 (14)
25	1.5 (5)
	3.0 (10)

Unidirectional wave spectra were generated based on Joint North Sea Wave Project (JONSWAP) parameters for the selected waves and used throughout the model investigation. Selected waves were defined as significant wave height, the average height of the highest one-third of the waves or H_s. In deep water, H_s is similar to H_{mo} (energy based wave) where $H_{mo} = 4\ (E)^{1/2}$, and E equals total energy in the spectra, which is obtained by integrating the energy density spectra over the frequency range.

Analysis of Model Data

Relative merits of the various plans were evaluated by the following:

a. Comparison of short-period wave heights and long-period wave heights (seiches) at selected locations in the model.

b. Comparison of wave-induced current patterns and magnitudes.

c. Comparison of tidal flows.

d. Visual observations.

In the wave-height data analysis, the average height of the highest one-third of the waves (H_s), recorded at each gauge location, was computed. All wave heights then were adjusted by application of Keulegan's equation[1] to compensate

[1] G. H. Keulegan (1950). "The gradual damping of a progressive oscillatory wave with distance in a prismatic rectangular channel," Unpublished data, National Bureau of Standards, Washington, DC, prepared at request of Director, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, by letter of May 1950.

for excessive model wave height attenuation due to viscous bottom friction. From this equation, reduction of model wave heights (relative to the prototype) can be calculated as a function of water depth, width of wave front, wave period, water viscosity, and distance of wave travel and the model data can be corrected and converted to their prototype equivalents.

Recent acquisition of National Data Buoy Center data from the Bering Sea near St. Paul Harbor as well as information obtained from Monitoring Completed Navigation Projects at the harbor (Bottin and Eisses 1997) indicate that wave periods as great as 25 sec can occur at the site. Since longer period spectral waves often induce more severe surf beat and the potential for seiches, previous studies were examined for seiche conditions. It was noted in reviewing earlier model experiments that long-period surges did occur in the harbor basin as a result of various frequencies in the spectral wave signals. Since oscillations occurred, it was considered important to obtain long-period wave information. Therefore, wave data obtained was filtered, and both short-period storm wave conditions as well as long-period wave conditions were analyzed and presented at the various gauge locations. In addition, wave-induced current velocities obtained in the model were the maximum that occurred during the wave spectra (usually occurring after a series of large waves in the wave signal and at long-period nodal points).

4 Experiments and Results

Experiments

Twelve study plans were evaluated during the initial portion of this investigation (Bottin 1996), and 15 plans were evaluated during the first reactivation of the model (Bottin and Acuff 1997). Therefore, plan numbering for this experimental series began with Plan No. 28. A 9.8-m-deep (32-ft-deep) draft entrance channel, an 8.8-m-deep (29-ft-deep) maneuvering area, a 3-m-deep (10-ft-deep) sediment trap, a 0.9-m-deep (3-ft-deep) connecting channel from the harbor to the salt lagoon, and a wave-dissipating spending beach inside the harbor (el 0.0 m (0.0 ft) with a +1.2 m (+4 ft) berm along its perimeter) were developed in previous studies and remained in the model for all experiments. Proposed improvement plans for this experimental series consisted of dredging a new small-boat channel and boat basin as well as installation of a shore-connected breakwater and an interior detached breakwater (for diversion of currents from the salt lagoon) in the existing harbor. Modifications also were made to the existing shoreline and depths in the existing harbor. Wave heights and wave-induced current patterns and magnitudes were obtained for variations in the harbor that consisted of changes in shoreline configurations, depths and/or structure lengths and alignments. Experiments of tidal flushing were conducted for changes in the orientation of the interior detached breakwater and depths in the harbor. Study plans that consisted of shoreline and depth changes in the harbor were expeditiously constructed in the model using gravel to determine optimum layouts. Brief descriptions of the small-boat harbor improvement plans are presented in the following subparagraphs, and dimensional details are shown in Plates 1-10.

a. Plan 28 (Plate 1) consisted of the installation of a 3.7-m-deep, 30.5-m-wide (12-ft-deep, 100-ft-wide) interior channel and a 3.7-m-deep (12-ft-deep) boat basin. It also included a 107-m-long (350-ft-long) interior shore-connected breakwater and a 45.7-m-long (150-ft-long) interior detached breakwater (both at els of +3 m (+10 ft)). The boat basin was revetted along its south and east sides. Slopes were 1V:1.25H on the shore-connected breakwater and 1V:1.5H on the interior detached breakwater and revetments. This configuration represents a 60-vessel boat basin.

b. Plan 29 (Plate 2) included the elements of Plan 28 with the shoreline configuration changed east of the boat basin. The shoreline was moved 33.5 m (110 ft) easterly in an arc and the revetment slope was changed to 1V:5H.

c. Plan 30 (Plate 2) entailed the elements of Plan 29 but the berm along the perimeter of the wave-dissipating spending beach was raised to +2.4 m (+8 ft).

d. Plan 31 (Plate 3) involved the elements of Plan 28, but the shoreline configuration east of the boat basin was moved 24.4 m (80 ft) in an arc and the revetment slope was changed to 1V:10H. The +2.4 m- (+8 ft-) berm el along the perimeter of the wave-dissipating spending beach was included.

e. Plan 32 (Plate 3) consisted of the elements of Plan 28 but the interior detached breakwater was extended 36.6 m (120 ft) in length. The structure was extended westerly 15.3 m (50 ft) on its original alignment and then angled 21.3 m (70 ft) southwesterly toward the channel. It also included the +2.4 m (+8 ft) berm along the perimeter of the spending beach.

f. Plan 33 (Plate 4) involved the elements of Plan 28, but the slope of the revetment east of the boat basin was changed to 1V:3H.

g. Plan 34 (Plate 5) entailed the elements of Plan 33, but 7.6 m (25 ft) was removed from the western end of the interior detached breakwater resulting in a 38.1-m-long (125-ft-long) structure. In addition, the area between the spending beach and interior detached breakwater was deepened to -1.5 m (-5 ft).

h. Plan 35 (Plate 6) included the elements of Plan 33, but the area between the spending beach and the interior detached breakwater was deepened to -0.9 m (-3 ft) and hardened (capped) with riprap to an el of -0.6 m (-2 ft).

i. Plan 36 (Plate 7) involved the elements of Plan 35 but the eastern 15.2-m (50-ft) portion of the interior detached breakwater was reoriented 45 deg southeasterly toward the small-boat basin.

j. Plan 37 (Plate 8) entailed the elements of Plan 36 but the interior channel was deepened to -4.9 m (-16 ft).

k. Plan 38 (Plate 9) involved the elements of Plan 37 but the area of the small-boat basin was reduced. The southeastern portion of the basin was filled. This configuration represents a 30-vessel basin.

l. Plan 39 (Plate 10) consisted of the elements of Plan 37 but the existing contours in an area west of the interior shore-connected breakwater were deepened (dredged) to an el of -6.7 m (-22 ft).

Wave height experiments

Wave height experiments were conducted for the initial and most promising improvement plans for the waves previously described in "Prototype wave data and selection of experimental waves," in Chapter 3. Experiments involving some proposed plans, however, were limited to the most critical wave conditions (i.e., 16-sec, 5.8-m (19-ft) waves). Wave gauge locations are shown on referenced plates.

Wave-induced current patterns and magnitudes

Wave-induced current patterns and magnitudes were obtained for selected improvement plans for various wave conditions. These experiments were conducted by timing the progress of a dye tracer relative to a known distance on the model surface at selected locations in the model.

Tidal flow experiments

Tidal flow experiments were conducted for selected improvement plans to determine flushing action throughout the harbor. Tidal current patterns and magnitudes were obtained with a dye tracer similar to those obtained for wave-induced currents.

Experimental Results

In analyzing results, the relative merits of various improvement plans were based on measured wave heights, wave-induced current patterns and magnitudes, and tidal flow currents. Model wave heights (significant wave heights or H_s) were tabulated to show measured values at selected locations. As indicated earlier, both short- and long-period (seiche) responses were analyzed and presented for the incident wave conditions at each gauge location. Wave-induced and tidal current patterns and magnitudes were shown on plates in the report.

Results of wave height experiments for Plan 28 are presented in Tables 1 and 2, respectively, for short- and long-period wave conditions with the +1.0-m (+3.2-ft) and +2.1-m (+7.0-ft) swls. Short-period wave conditions were those obtained with periods of 30 sec or less. Long-period conditions were those over 30 sec and represented seiche produced by the wave spectra. For short-period wave conditions, maximum wave heights[1] were 0.55 m (1.8 ft) in the interior entrance channel (Gauge 5) and 0.18 m (0.6 ft) in the small-boat harbor mooring area (Gauge 7) for 16-sec, 5.8-m (19-ft) waves with the +2.1-m (+7.0-ft) swl. For long-period wave conditions, maximum wave heights were 0.8 m (2.6 ft) in the interior channel for 16-sec, 5.8-m (19-ft) waves with the +2.1-m (+7.0-ft) swl, and 0.7 m (2.3 ft) in the mooring area for 16-sec, 5.8-m (19-ft) waves with

[1] Refers to maximum significant wave heights throughout report.

the +1.0-m (+3.2-ft) swl. All short-period wave heights in the mooring area were within the generally accepted 0.3-m (1.0-ft) wave height criterion for small-boat harbors. In several instances, long-period oscillations resulted in wave heights over 0.6 m (2.0-ft). These heights, when associated with long-period waves, generally do not result in vessel damage; however, horizontal currents between nodes and antinodes in a standing wave system may result in undesirable mooring conditions. Damage to vessel and floating docks generally are not a major problem when moorings are properly oriented and vessels properly moored. Wave periods associated with the maximum long-period (surge) conditions in the mooring area ranged from 112 to 142 sec.

A comparison of short- and long-period wave conditions for Plans 28-32 is shown in Table 3 for 16-sec, 5.8-m (19-ft) waves with the +1.0-m (+3.2-ft) swl. For short-period conditions, maximum wave heights were 0.43 m (1.4 ft) in the interior entrance channel (Gauge 5) for Plans 28, 29, 30, and 32, and 0.15 m (0.5 ft) in the small-boat mooring area (Gauge 8) for Plans 28, 29, and 31. Maximum wave heights, for long-period conditions, were 0.7 m (2.3 ft) in the entrance channel for Plans 28, 29, and 32 and 0.7 m (2.3 ft) in the mooring area for Plan 28. Short-period conditions resulted in wave heights that were satisfactory for all these plans. It was noted that basin modifications and/or the extension of the interior detached breakwater had little effect, however, on long-period wave heights in the harbor. Wave periods associated with the maximum long-period (surge) conditions in the mooring area ranged from 111 to 144 sec.

Wave heights obtained for representative waves for Plan 32 are presented in Tables 4 and 5, respectively, for short- and long-period wave conditions with the +1.0-m (+3.2-ft) and +2.1-m (+7.0-ft) swls. For short-period wave conditions, maximum wave heights were 0.64 m (2.1 ft) in the interior entrance channel (Gauge 5) and 0.18 m (0.6 ft) in the small-boat mooring area (Gauge 8) for 16-sec, 5.8-m (19-ft) and 20-sec, 4.3-m (14-ft) waves with the +2.1-m (+7.0-ft) swl. For long-period wave conditions, maximum wave heights were 0.91 m (3.0 ft) in the interior channel and 0.76 m (2.5 ft) in the mooring area for 16-sec, 5.8-m (19-ft) waves with the +2.1-m (+7.0-ft) swl. Wave periods associated with the maximum long-period (surge) conditions in the mooring area ranged from 121 to 139 sec.

Wave-induced current patterns and magnitudes obtained for representative waves with Plan 32 installed in the model are presented in Plates 11 and 12, respectively, for the +1.0-m (+3.2-ft) and +2.1-m (+7.0-ft) swls. Maximum velocities were 3.38 m/s (11.1 fps) in the area between the spending beach and the interior detached breakwater, 3.0 m/s (9.7 fps) in the interior entrance channel, and 1.92 m/s (6.3 fps) on the east side of the head of the shore-connected breakwater. These maximum velocities all occurred for 16-sec, 5.8-m (19-ft) conditions.

Wave-induced current patterns and magnitudes obtained for Plans 33-35 for 16-sec, 5.8-m (19-ft) waves with the +1.0 m (+3.2-ft) swl are presented in Plate 13. Maximum velocities were 2.26 m/s (7.4 fps) in the area between the spending beach and the interior detached breakwater for Plan 35, 1.62 m/s (5.3 fps) in the interior entrance channel for Plan 33, and 1.52 m/s (5.0 fps) on

the east side of the head of the shore-connected breakwater for Plan 34. Plan 35 appeared to be best in regard to current velocities in the channel and mooring area. Due to excessive velocities obtained in the area between the interior detached breakwater and the spending beach, it appears the area should be hardened (covered with riprap) to prevent scour. Excessive current velocities obtained in the interior channel also may pose navigation problems for extreme storm wave events. In addition, strong wave-induced currents along the east side of the head of the interior shore-connected breakwater could cause problems for vessels moored in this vicinity. These values also indicate that toe protection of the breakwater head may be required.

A comparison of short- and long-period wave conditions for Plans 33-35 is shown in Table 6 for 16-sec, 5.8-m (19-ft) waves with the +1.0-m (+3.2-ft) swl. For short-period conditions, maximum wave heights were 0.4 m (1.3 ft) in the interior entrance channel (Gauge 5) for Plans 34 and 35, and 0.12 m (0.4 ft) in the small-boat mooring area (Gauge 8) for Plans 33, 34, and 35. Maximum wave heights for long-period wave conditions were 0.7 m (2.3 ft) in the entrance channel for Plans 34 and 35, and 0.82 m (2.7 ft) in the mooring area for Plan 34. Short-period conditions for Plans 33-35 resulted in satisfactory wave heights in the small-boat harbor. For long-period waves, Plan 35 (the -0.6-m (-2-ft) el between the spending beach and interior detached breakwater) resulted in reduced wave heights, with respect to waves in the mooring areas, versus the other plans. Wave periods associated with the maximum long-period (surge) conditions in the mooring area ranged from 126 to 140 sec.

Ebb tidal current patterns were secured for Plans 35 and 36 and are shown in Plate 14 for the 2.1-m (7.0-ft) tidal range. The current patterns were similar for both plans, but visual observations indicated a better distribution of flow in the small-boat harbor basin with the angled interior detached breakwater of Plan 36. Magnitudes obtained for Plan 36 (also shown in Plate 14) were 0.85 m/s (2.8 fps) on each side of the interior detached breakwater, 0.3 m/s (0.9 fps) in the small-boat basin, and 0.18 m/s (0.6 fps) in the interior channel. Magnitudes were obtained during the midrange of the tidal cycle.

Ebb tidal current patterns and magnitudes secured for Plan 37 are shown in Plate 15 both with and without waves for the 2.1-m (7.0-ft) tidal range. Results shown in Plate 15 with waves included typical, everyday wave conditions of about 1.2 to 1.8 m (4 to 6 ft). Without waves, magnitudes obtained were 0.82 and 0.76 m/s (2.7 and 2.5 fps), respectively, east and west of the angled interior detached breakwater, 0.3 m/s (1.0 fps) in the small-boat basin, and 0.18 m/s (0.6 fps) in the interior channel. Magnitudes obtained for Plan 37, with waves, were 0.7 and 0.82 m/s (2.3 and 2.7 fps), respectively, east and west of the interior breakwater, 0.21 m/s (0.7 fps) in the small-boat basin, and 0.27 m/s (0.9 fps) in the interior channel. It was noted that wave conditions improved harbor circulation. Typical wave conditions resulted in increased currents out the main entrance of the harbor, whereas without waves, tidal flows moved out of the harbor predominantly northerly along the shoreline through the -3.0-m-deep (-10-ft-deep) deposition basin.

Wave heights obtained with representative wave conditions for Plan 37 are presented in Tables 7 and 8, respectively, for short- and long-period waves with the +1.0-m (+3.2-ft) and +2.1-m (+7.0-ft) swls. For short-period wave conditions, maximum wave heights were 0.46 m (1.5 ft) in the interior entrance channel (Gauge 5) and 0.21 m (0.7 ft) in the small-boat mooring area (Gauge 8) for 16-sec, 5.8-m (19-ft) waves with the +2.1-m (+7.0-ft) swl. For long-period wave conditions, maximum wave heights were 0.82 m (2.7 ft) in the interior channel for 16-sec, 5.8-m (19-ft) waves and 0.82 m (2.7 ft) in the mooring area for 20-sec, 4.3-m (14-ft) waves with the +2.1-m (+7.0-ft) swl. Wave periods associated with the maximum long-period (surge) conditions in the mooring area ranged from 115 to 132 sec.

Wave-induced current patterns and magnitudes obtained for representative waves with Plan 37 installed in the model are presented in Plates 16 and 17, respectively, for the +1.0-m (+3.2-ft) and +2.1-m (+7.0-ft) swls. Maximum velocities were 2.35 m/s (7.7 fps) in the area between the spending beach and the interior detached breakwater for 16-sec, 5.8-m (19-ft) waves, 1.68 m/s (5.5 fps) in the interior entrance channel for 16-sec, 5.8-m (19-ft) and 20-sec, 4.3-m (14-ft) waves, and 1.68 m/s (5.5 fps) on the east side of the head of the shore-connected breakwater for 16-sec, 5.8-m (19-ft) waves. All maximum velocities occurred for the +1.0-m (+3.2-ft) swl. Visual observations with Plan 37 installed, versus the earlier plans, revealed that the -4.9-m (-16-ft) deep channel enhanced circulation in the small-boat basin. The plan resulted in slightly stronger eddies in the basin.

Evaluation of results at this point in the investigation indicated that the layout of Plan 37 was the optimum 60-vessel configuration considering wave and surge conditions in the small-boat harbor mooring area and harbor circulation (wave-induced current patterns and magnitudes and ebb tidal flow conditions).

Wave heights obtained with Plan 38 installed for representative wave conditions are presented in Tables 9 and 10, respectively, for short- and long-period waves with the +1.0-m (+3.2-ft) and +2.1-m (+7.0-ft) swls. For short-period wave conditions, maximum wave heights were 0.52 m (1.7 ft) in the interior entrance channel (Gauge 5) and 0.21 m (0.7 ft) in the small-boat mooring area (Gauge 8) for 16-sec, 5.8-m (19-ft) waves with the +2.1-m (+7.0-ft) swl. For long-period wave conditions, maximum wave heights were 0.82 m (2.7 ft) in the interior channel and 0.91 m (3.0 ft) in the mooring area for 16-sec, 5.8-m (19-ft) waves with the +2.1-m (+7.0-ft) swl. Wave periods associated with the maximum long-period (surge) conditions in the mooring area ranged from from 118 to 139 sec.

Wave-induced current patterns and magnitudes obtained for Plan 38 are presented in Plates 18 and 19 for representative waves for the +1.0-m (+3.2-ft) and +2.1-m (+7.0-ft) swls. Maximum velocities were 2.44 m/s (8.0 fps) in the area between the spending beach and the interior detached breakwater for 16-sec, 5.8-m (19-ft) waves with the +1.0-m (+3.2-ft) swl, 2.04 m/s (6.7 fps) in the interior entrance channel for 16-sec, 5.8-m (19-ft) waves with the +2.1-m (+7.0-ft) swl, and 1.46 m/s (4.8 fps) on the east side of the head of the shore-

connected breakwater for 20-sec, 4.3-m (14-ft) waves with the +1.0-m (+3.2-ft) swl.

Ebb tidal current patterns and magnitudes for Plan 38 are presented in Plate 20 for the +2.1-m (+7.0-ft) tide range. Magnitudes obtained were 0.98 and 0.88 m/s (3.2 and 2.9 fps), respectively, east and west of the angled interior detached breakwater, 0.34 m/s (1.1 fps) in the small-boat basin, and 0.12 m/s (0.4 fps) in the interior channel. These tidal flow patterns and magnitudes were similar to those obtained for Plan 37.

Wave heights obtained for representative wave conditions for Plan 39 are presented in Tables 11 and 12, respectively, for short- and long- period waves with the +1.0-m (+3.2-ft) and +2.1-m (+7.0-ft) swls. For short-period wave conditions, maximum wave heights were 0.52 m (1.7 ft) in the interior entrance channel (Gauge 5) and 0.21 m (0.7 ft) in the small-boat mooring area (Gauges 8 and 9) for 16-sec, 5.8-m (19-ft) waves with the +2.1-m (+7.0-ft) swl. For long-period wave conditions, maximum wave heights were 0.79 m (2.6 ft) in the interior channel for 16-sec, 5.8-m (19-ft) waves and 0.91 m (3.0 ft) in the mooring area for 16-sec, 5.8-m (19-ft) and 20-sec, 4.3-m (14-ft) waves with the +2.1-m (+7.0-ft) swl. Wave periods associated with the maximum long-period (surge) conditions in the mooring area ranged from 115 and 130 sec.

Wave-induced current patterns and magnitudes secured for representative waves with Plan 39 installed are presented in Plates 21 and 22, respectively, for the +1.0-m (+3.2-ft) and +2.1-m (+7.0-ft) swls. Maximum velocities were 2.38 m/s (7.8 fps) in the area between the spending beach and the interior detached breakwater for 16-sec, 5.8-m (19-ft) waves with the +1.0-m (+3.2-ft) swl, 1.7 m/s (5.6 fps) in the interior entrance channel for 16-sec, 5.8-m (19-ft) waves with the +2.1-m (+7.0-ft) swl, and 1.46 m/s (4.8 fps) on the east side of the head of the shore-connected breakwater for 16-sec, 5.8-m (19-ft) waves with the +1.0-m (+3.2 ft) swl.

Ebb tidal current patterns and magnitudes secured for Plan 39 are shown in Plate 23 for the +2.1-m (+7.0-ft) tidal range. Magnitudes were 0.85 m/s (2.8 fps) on each side of the angled interior detached breakwater, 0.3 m/s (1.0 fps) in the small-boat basin, and 0.18 m/s (0.6 fps) in the interior channel. These tidal flow patterns were similar to those obtained for Plans 37 and 38.

During the conduct of the investigation, all improvement plans experienced long-period (surge) conditions in the small-boat harbor mooring area. These surges (heights) generally had amplitudes ranging from about 0.6 to 0.9 m (2 to 3 ft) with associated periods ranging from approximately 110 to 145 sec (depending on the plan). These vertical wave heights generally do not cause problems, or result in vessel damage, in small-boat harbors. The horizontal velocities associated with a standing wave system, however, could pose problems for floating dock systems and vessels. Therefore, it is important that these horizontal movements be considered in the small-boat harbor design to ensure proper orientation and anchorage of dock systems as well as proper orientation and mooring of vessels. Current data obtained that was associated with harbor seiching is presented in Plate 24 for the 60-vessel harbor configuration. The

vectors depict directions of the back and forth (horizontal) current movements in the mooring area. Maximum velocities obtained ranged from 0.21 to 0.3 m/s (0.7 to 1.0 fps) depending on location.

In earlier studies, experiments were conducted with the wave-dissipating spending beach inside the main harbor constructed to an el of +3.7 m (+12 ft). Experiments conducted for this series of improvement plans indicated that the spending beach could be reduced to an el of 0.0 m (0.0 ft), (with a +1.2-m (+4.0-ft) berm along its perimeter) and still provide essentially the same level of protection from storm wave conditions in the small-boat harbor.

Also in earlier studies, experiments were conducted with the salt lagoon channel molded in with its natural depths and geometry. For this study, the channel was modeled with its proposed dredged depth of -0.9 m (-3-ft) with a bottom width of 12.2 m (40 ft) and 1V:3H side slopes. The first 61 m (200 ft) was armored with riprap on the south bank and the modified channel alignment was included in the model. The proposed salt lagoon channel geometry was equally effective in driving circulation in the small-boat harbor as in previous studies.

5 Conclusions

Based on results of the coastal model investigation reported herein, the conclusions are as follows:

a. Preliminary experiments indicated that all improvement plans would result in wave heights of less than 0.3 m (1.0 ft) in the small-boat mooring area for short-period storm wave conditions.

b. Preliminary experiments indicated that the harbor would experience long-period (surge) conditions for all improvement plans.

c. Preliminary experiments indicated that the area between the wave-dissipating spending beach and the interior detached breakwater should be constructed to an el of -0.6 m (-2.0 ft) to reduce wave heights in the small-boat harbor mooring areas. Excessive wave-induced currents in this area, however, indicated that the area should be hardened (capped with riprap) to prevent scour.

d. Preliminary experiments indicated that strong wave-induced currents in the interior channel may cause navigation difficulties during extreme storm wave events. Strong wave-induced currents along the area east of the shore-connected breakwater also may pose problems for vessels mooring in this vicinity. These current magnitudes also indicate that toe protection at the head of the structure may be required.

e. Preliminary experiments indicated that the angled interior detached breakwater would result in enhanced circulation and better distribution of flow in the small-boat harbor basin for ebb tidal currents as opposed to the straight structure.

f. Preliminary experiments indicated that the -4.9-m-deep (-16-ft-deep) interior channel would result in enhanced wave-induced circulation and stronger eddies in the small-boat basin as opposed to the -3.7-m-deep (-12-ft-deep) channel.

g. Experiments indicated that the 60-vessel plan configuration (Plan 37) will provide adequate wave and surge protection to the small-boat harbor as well as adequate harbor circulation.

h. Experiments indicated that the 30-vessel plan configuration (Plan 38) will provide adequate wave and surge protection to the small-boat harbor as well as adequate harbor circulation.

i. Experiments indicated that an increase in depths in the harbor to -6.7 m (-22 ft) west of the interior shore-connected breakwater (Plan 39) will have no negative impacts on wave and surge conditions or harbor circulation in the small-boat harbor.

j. Experiments indicated that long-period surge conditions exist in the harbor during storm wave events. These conditions must be properly designed for with respect to the dock systems and vessel moorage orientation in the small-boat harbor.

k. Experiments indicated that the 0.0-m (0.0-ft) el of the wave-dissipating spending beach (with the +1.2-m (+4.0-ft) berm along its perimenter) assessed during the study will provide essentially the same level of protection from storm waves in the mooring area as the +3.7-m (+12.0-ft) el spending beach tested in earlier studies.

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Table 1
Short-Period Wave Heights for Plan 28

Experimental Wave		Wave Height at Indicated Gauge Location, ft[1]										
Period (sec)	Height (ft)	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11
swl = +3.2 ft												
8	10	2.3	0.6	1.1	0.3	0.6	0.3	0.1	0.2	0.2	0.2	0.7
10	10	2.6	0.7	0.9	0.5	0.8	0.4	0.2	0.2	0.3	0.2	0.9
16	14.4	3.5	1.2	1.5	1.4	1.3	0.9	0.3	0.4	0.5	0.4	1.3
16	19	4.2	1.4	1.7	1.6	1.4	1.0	0.4	0.5	0.6	0.5	1.4
20	14	4.0	1.4	1.7	1.5	1.4	0.9	0.4	0.5	0.6	0.5	1.5
25	5	2.0	1.0	1.7	1.3	1.2	0.6	0.1	0.2	0.3	0.2	0.6
swl = +7.0 ft												
8	10	2.8	1.0	1.0	0.8	0.7	0.5	0.2	0.2	0.3	0.3	0.7
10	10	3.5	1.0	1.1	0.8	0.9	0.7	0.3	0.3	0.4	0.3	1.0
16	14.4	4.6	1.6	2.0	1.6	1.4	1.1	0.5	0.5	0.7	0.6	1.8
16	19	5.6	2.0	2.6	1.9	1.8	1.3	0.6	0.6	0.8	0.7	2.2
20	14	4.8	1.7	2.2	1.8	1.6	1.1	0.5	0.6	0.7	0.6	2.0
25	5	2.5	0.8	1.1	1.1	1.2	0.9	0.2	0.3	0.4	0.3	0.9

[1] To convert feet to meters, multiply by 0.3048.

Table 2
Long-Period Wave Heights for Plan 28

Experimental Wave		Wave Height at Indicated Gauge Location, ft										
Period (sec)	Height (ft)	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11
swl = +3.2 ft												
8	10	2.4	0.7	1.2	0.5	0.8	0.6	0.7	0.9	0.6	0.6	1.0
10	10	2.9	1.0	1.3	1.1	1.3	1.1	1.1	1.3	0.8	0.9	1.4
16	14.4	4.0	1.7	2.2	2.4	2.0	1.8	1.7	2.0	1.6	1.6	2.0
16	19	4.7	2.0	2.4	2.6	2.3	1.9	2.3	1.8	1.6	1.8	2.2
20	14	4.5	2.0	2.4	2.4	2.4	1.8	1.9	2.0	1.4	1.6	2.4
25	5	2.1	0.7	1.1	1.0	0.9	0.6	0.5	0.7	0.6	0.6	0.8
Swl = +7.0 ft												
8	10	2.8	1.1	1.1	0.9	0.9	0.8	0.9	1.1	0.6	0.7	1.1
10	10	3.6	1.2	1.3	1.2	1.3	1.1	1.2	1.2	0.9	1.0	1.4
16	14.4	5.0	2.0	2.6	2.5	2.2	1.7	1.9	2.1	1.5	1.8	2.6
16	19	6.1	2.6	3.1	3.1	2.6	1.9	2.0	2.1	1.7	1.7	3.2
20	14	5.3	2.2	2.9	2.9	2.5	1.9	2.0	2.2	1.6	1.8	2.8
25	5	2.6	0.9	1.2	1.2	0.9	0.7	0.6	0.7	0.6	0.6	1.0

Table 3
Comparison of Wave Heights for Plans 28-32; 16-sec, 19-ft waves; swl = +3.2 ft

Plan	Wave Height at Indicated Gauge Location, ft										
	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11
Short-Period Wave Conditions											
28	4.2	1.4	1.7	1.6	1.4	1.0	0.4	0.5	0.6	0.5	1.4
29	4.2	1.3	1.7	1.5	1.4	0.9	0.3	0.5	0.6	0.5	1.4
30	4.0	1.4	1.7	1.5	1.4	0.9	0.4	0.4	0.6	0.5	1.3
31	4.1	1.3	1.8	1.6	1.3	0.9	0.4	0.5	0.5	0.5	1.4
32	4.5	1.4	1.8	1.6	1.4	0.9	0.3	0.4	0.5	0.5	1.4
Long-Period Wave Conditions											
28	4.7	2.0	2.4	2.6	2.3	1.9	2.3	1.8	1.6	1.8	2.2
29	4.6	1.9	2.4	2.3	2.3	1.6	1.9	1.8	1.4	1.6	2.1
30	4.6	1.9	2.5	2.3	2.1	1.7	1.8	1.5	1.4	1.5	2.1
31	4.7	1.9	2.4	2.4	2.1	1.6	1.9	1.7	1.4	1.7	2.2
32	5.0	2.0	2.7	2.6	2.3	1.6	1.9	1.6	1.4	1.6	2.1

Table 4
Short-Period Wave Heights for Plan 32

Experimental Wave		Wave Height at Indicated Gauge Location, ft										
Period (sec)	Height (ft)	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11
swl = +3.2 ft												
10	10	2.5	0.7	0.9	0.7	0.7	0.5	0.2	0.2	0.3	0.2	0.9
16	19	4.5	1.4	1.8	1.6	1.4	0.9	0.3	0.4	0.5	0.5	1.4
20	14	4.0	1.3	1.8	1.5	1.5	0.9	0.3	0.3	0.5	0.4	1.3
25	10	3.3	1.1	1.6	1.5	1.2	0.8	0.3	0.3	0.4	0.4	1.1
swl = +7.0 ft												
10	10	3.3	1.0	0.7	0.6	1.1	0.6	0.2	0.3	0.4	0.3	1.0
16	19	5.0	1.8	1.8	1.7	2.1	1.0	0.4	0.6	0.7	0.6	2.0
20	14	4.6	1.6	1.7	1.7	1.5	1.0	0.4	0.6	0.6	0.6	1.9
25	10	4.4	1.4	1.6	1.8	1.5	0.9	0.3	0.4	0.6	0.5	1.6

Table 5
Long-Period Wave Heights for Plan 32

Experimental Wave		Wave Height at Indicated Gauge Location, ft										
Period (sec)	Height (ft)	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11
swl = +3.2 ft												
10	10	2.7	1.0	1.4	1.2	1.2	1.0	1.1	1.2	0.8	0.9	1.3
16	19	5.0	2.0	2.7	2.6	2.3	1.6	1.9	1.6	1.4	1.6	2.1
20	14	4.4	1.9	2.6	2.4	2.5	1.6	1.7	1.4	1.3	1.5	2.2
25	10	3.7	1.5	2.2	2.1	1.8	1.3	1.4	1.7	1.2	1.4	1.6
swl = +7.0 ft												
10	10	3.4	1.2	1.1	1.1	1.5	0.9	1.2	1.1	0.7	0.9	1.4
16	19	5.6	2.5	2.6	3.0	3.0	1.7	2.5	2.1	1.5	1.7	2.8
20	14	5.1	2.2	2.5	2.7	2.4	1.6	2.0	2.0	1.5	1.7	2.8
25	10	4.7	1.8	2.1	2.4	2.1	1.5	1.5	1.5	1.2	1.3	2.2

Table 6
Comparison of Wave Heights for Plans 33-35; 16-sec, 19-ft waves; swl = +3.2 ft

Plan	Wave Height at Indicated Gauge Location, ft										
	Gauge 1	Gauge 2	Gauge 3A	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11
					Short-Period Wave Conditions						
33	4.3	1.4	0.5	1.6	1.2	0.8	0.3	0.4	0.5	0.4	1.5
34	4.3	1.4	0.5	1.6	1.3	0.9	0.3	0.4	0.5	0.4	1.6
35	4.2	1.3	0.5	1.6	1.3	0.9	0.3	0.4	0.5	0.4	1.5
					Long-Period Wave Conditions						
33	4.8	2.1	2.9	2.5	2.2	2.1	2.4	2.2	1.9	1.7	2.5
34	4.7	2.0	3.2	2.8	2.3	1.6	2.3	2.7	1.9	1.9	2.3
35	4.6	1.9	3.1	2.6	2.3	1.8	2.0	2.3	1.5	1.9	2.3

Table 7
Short-Period Wave Heights for Plan 37

Experimental Wave		Wave Height at Indicated Gauge Location, ft										
Period (sec)	Height (ft)	Gauge 1	Gauge 2	Gauge 3A	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11
swl = +3.2 ft												
10	10	2.1	0.6	0.3	0.7	0.5	0.3	0.2	0.3	0.2	0.2	0.6
16	19	4.2	1.4	0.6	1.6	1.0	0.8	0.4	0.4	0.6	0.4	1.5
20	14	3.5	1.2	0.5	1.5	1.0	0.7	0.4	0.4	0.4	0.4	1.1
25	10	3.1	0.9	0.4	1.5	0.9	0.6	0.2	0.3	0.4	0.3	0.9
swl = +7.0 ft												
10	10	3.7	1.0	0.3	0.8	0.6	0.3	0.3	0.3	0.5	0.3	0.8
16	19	5.4	1.8	0.7	1.8	1.5	0.9	0.6	0.7	0.8	0.6	2.3
20	14	4.8	1.7	0.6	1.8	1.4	0.8	0.6	0.6	0.8	0.6	2.0
25	10	4.5	1.5	0.5	2.0	1.2	0.6	0.5	0.5	0.7	0.5	1.7

Table 8
Long-Period Wave Heights for Plan 37

Experimental Wave		Wave Height at Indicated Gauge Location, ft										
Period (sec)	Height (ft)	Gauge 1	Gauge 2	Gauge 3A	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11
swl = +3.2 ft												
10	10	2.5	0.9	2.1	1.3	1.0	1.0	1.2	1.2	0.8	0.9	1.2
16	19	4.7	1.9	3.4	2.7	2.0	1.9	2.2	2.2	1.6	2.0	2.3
20	14	4.1	1.8	3.0	2.4	2.0	1.7	1.9	2.0	1.3	1.6	2.0
25	10	3.5	1.3	2.6	2.3	1.6	1.4	1.9	1.8	1.8	1.6	1.4
swl = +7.0 ft												
10	10	3.7	1.2	2.2	1.3	1.3	1.1	1.3	1.4	0.9	1.0	1.2
16	19	5.8	2.4	4.0	3.1	2.7	1.8	2.4	2.4	1.9	2.1	3.5
20	14	5.3	2.3	4.0	2.9	2.5	1.9	2.4	2.7	1.8	2.0	2.8
25	10	4.8	1.9	2.9	2.7	1.9	1.5	2.0	2.1	1.5	1.7	2.3

Table 9
Short-Period Wave Heights for Plan 38

Experimental Wave		Wave Height at Indicated Gauge Location, ft										
Period (sec)	Height (ft)	Gauge 1	Gauge 2	Gauge 3B	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11A
swl = +3.2 ft												
10	10	2.8	0.8	0.3	0.8	0.6	0.3	0.2	0.2	0.4	0.3	0.7
16	19	4.0	1.3	0.5	1.6	1.1	0.6	0.3	0.5	0.6	0.4	1.6
20	14	3.8	1.3	0.5	1.5	1.2	0.6	0.3	0.5	0.5	0.4	1.5
25	10	3.4	0.9	0.4	1.4	1.0	0.5	0.2	0.4	0.5	0.3	1.5
swl = +7.0 ft												
10	10	3.8	1.1	0.3	0.9	0.8	0.5	0.3	0.3	0.6	0.4	0.7
16	19	5.5	1.9	0.7	1.9	1.7	1.0	0.6	0.7	0.9	0.7	1.9
20	14	4.8	1.7	0.6	1.8	1.4	0.9	0.5	0.6	0.8	0.7	1.8
25	10	4.5	1.5	0.6	2.0	1.3	0.8	0.5	0.5	0.8	0.6	1.8

Table 10
Long-Period Wave Heights for Plan 38

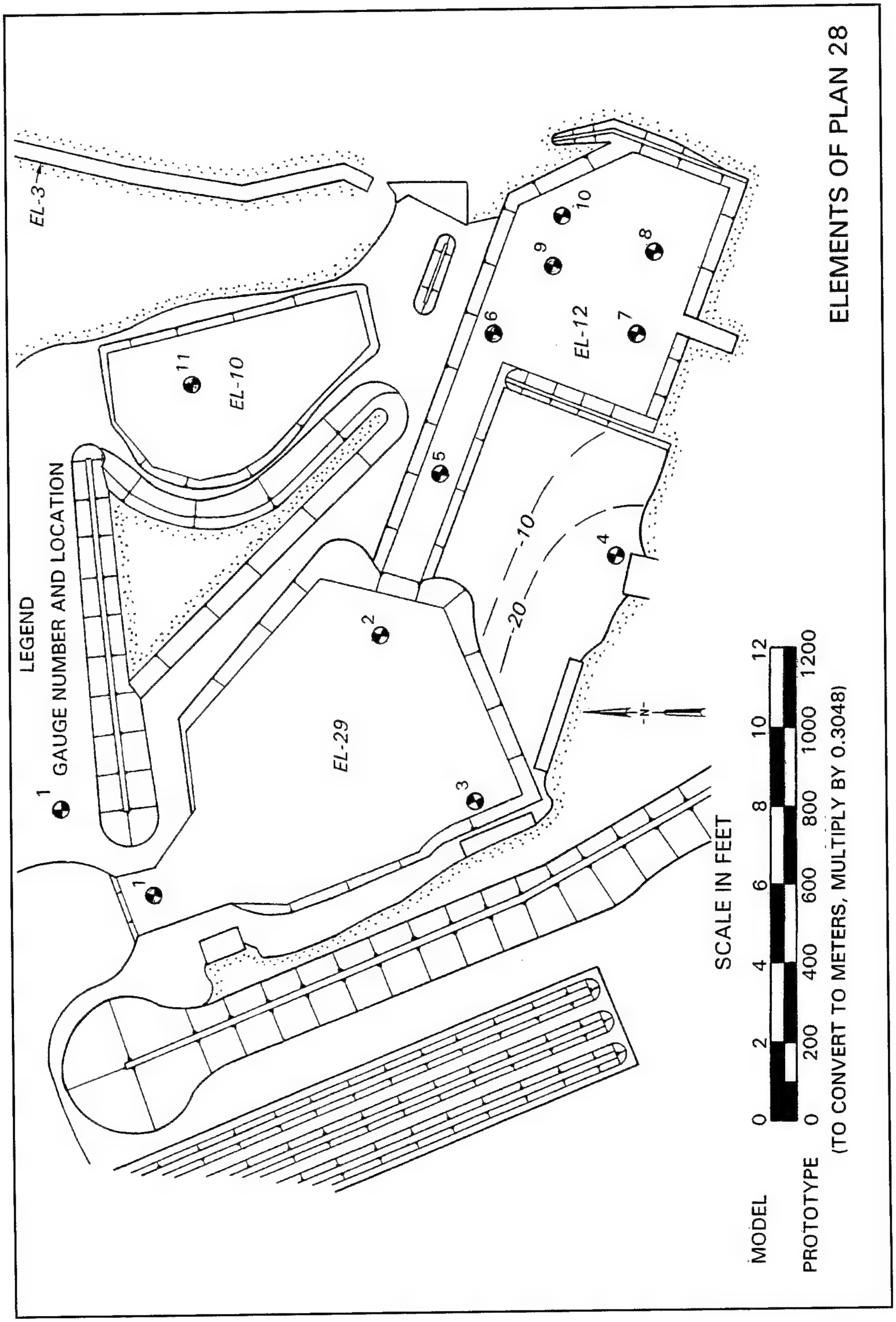
Experimental Wave		Wave Height at Indicated Gauge Location, ft										
Period (sec)	Height (ft)	Gauge 1	Gauge 2	Gauge 3B	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11A
swl = +3.2 ft												
10	10	3.2	1.0	2.0	1.4	1.2	1.0	1.6	1.6	1.0	1.4	2.5
16	19	4.6	1.9	3.0	2.6	2.0	1.9	2.1	2.3	1.7	2.2	4.6
20	14	4.3	1.8	2.7	2.4	2.1	1.5	1.8	2.0	1.4	2.2	4.3
25	10	3.8	1.3	2.4	2.2	1.8	1.4	1.8	1.9	1.5	1.8	3.6
swl = +7.0 ft												
10	10	3.9	1.3	1.9	1.4	1.2	1.0	1.4	1.6	1.0	1.2	2.3
16	19	6.0	2.5	3.7	3.3	2.7	1.8	2.7	3.0	1.9	2.6	5.4
20	14	5.3	2.2	3.6	3.0	2.6	1.8	2.5	2.9	1.9	2.5	5.1
25	10	4.9	2.0	2.7	2.8	2.0	1.6	2.1	2.3	1.6	2.0	3.9

Table 11
Short-Period Wave Heights for Plan 39

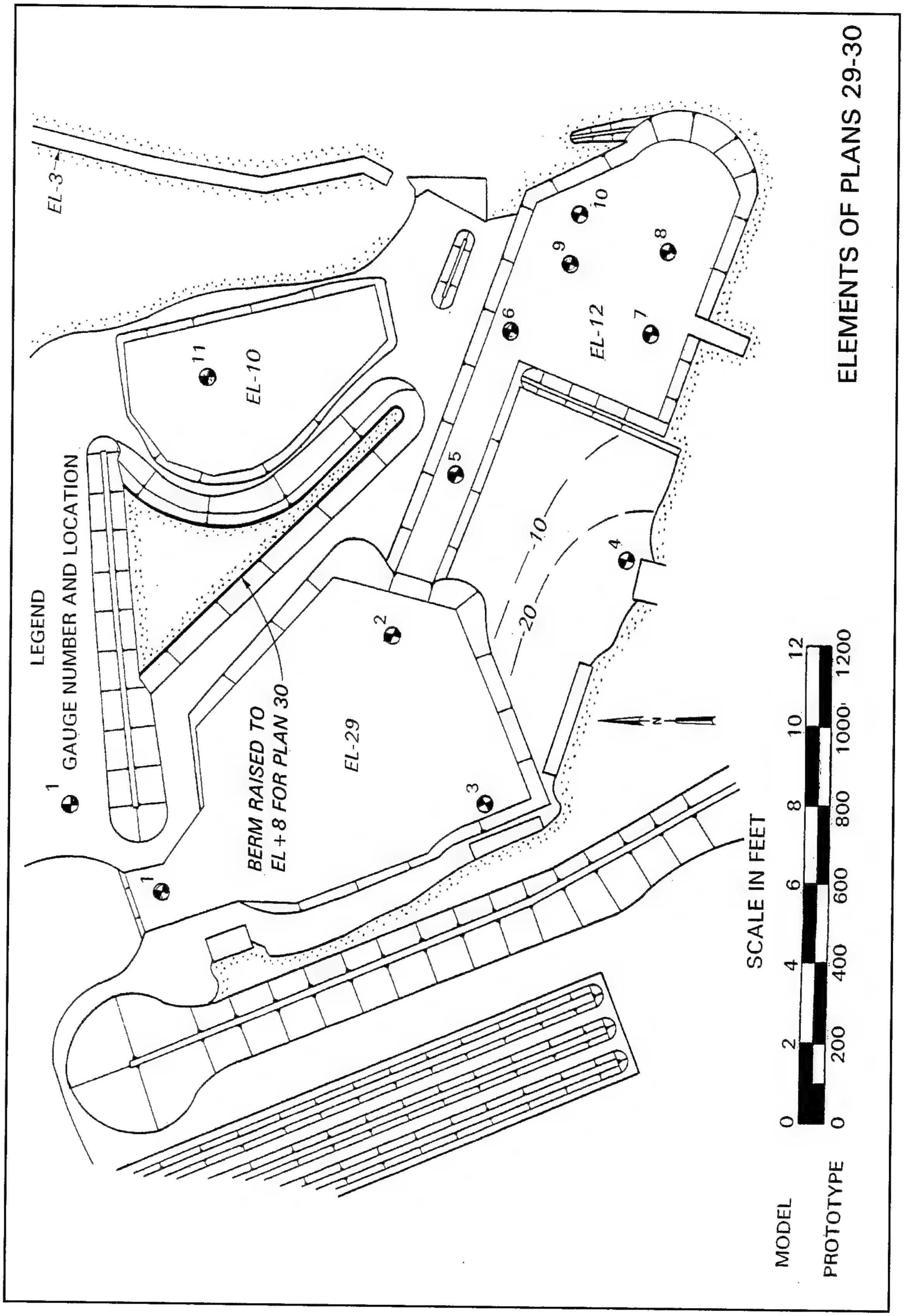
Experimental Wave		Wave Height at Indicated Gauge Location, ft										
Period (sec)	Height (ft)	Gauge 1	Gauge 2	Gauge 3A	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11A
swl = +3.2 ft												
10	10	2.2	0.6	0.3	0.8	0.4	0.3	0.2	0.2	0.2	0.2	0.8
16	19	4.1	1.2	0.6	1.8	1.0	0.7	0.4	0.4	0.5	0.4	1.7
20	14	3.9	1.2	0.6	1.7	1.0	0.6	0.4	0.4	0.5	0.4	1.7
25	10	3.2	0.8	1.0	1.5	0.7	0.5	0.3	0.3	0.4	0.3	1.6
swl = +7.0 ft												
10	10	3.6	1.0	1.0	1.3	0.8	0.4	0.2	0.3	0.3	0.3	0.8
16	19	5.8	2.0	1.2	2.4	1.7	1.1	0.7	0.7	0.8	0.6	1.8
20	14	4.9	1.6	1.4	2.2	1.5	0.9	0.6	0.6	0.7	0.5	1.7
25	10	4.6	1.4	1.0	2.2	1.2	0.7	0.5	0.5	0.6	0.4	1.7

Table 12
Long-Period Wave Heights for Plan 39

Experimental Wave		Wave Height at Indicated Gauge Location, ft										
Period (sec)	Height (ft)	Gauge 1	Gauge 2	Gauge 3A	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11A
swl = +3.2 ft												
10	10	2.4	0.9	2.2	1.3	1.0	1.3	1.3	1.5	1.2	1.3	2.4
16	19	4.6	1.7	3.5	2.7	2.0	2.0	1.9	2.5	1.8	2.1	4.3
20	14	4.3	1.7	3.6	2.6	1.9	1.8	2.0	2.4	1.5	1.9	4.2
25	10	3.5	1.2	3.6	2.4	1.5	1.7	2.1	2.2	1.6	2.0	3.3
swl = +7.0 ft												
10	10	3.8	1.2	3.2	1.6	1.1	1.2	1.5	1.5	1.0	1.2	1.9
16	19	6.4	2.7	4.1	3.8	2.6	2.1	2.7	3.0	2.1	2.5	4.9
20	14	5.6	2.3	4.1	3.2	2.3	2.1	2.9	3.0	2.1	2.4	4.6
25	10	5.0	1.9	3.6	3.0	1.8	1.7	2.2	2.1	1.6	1.9	3.5

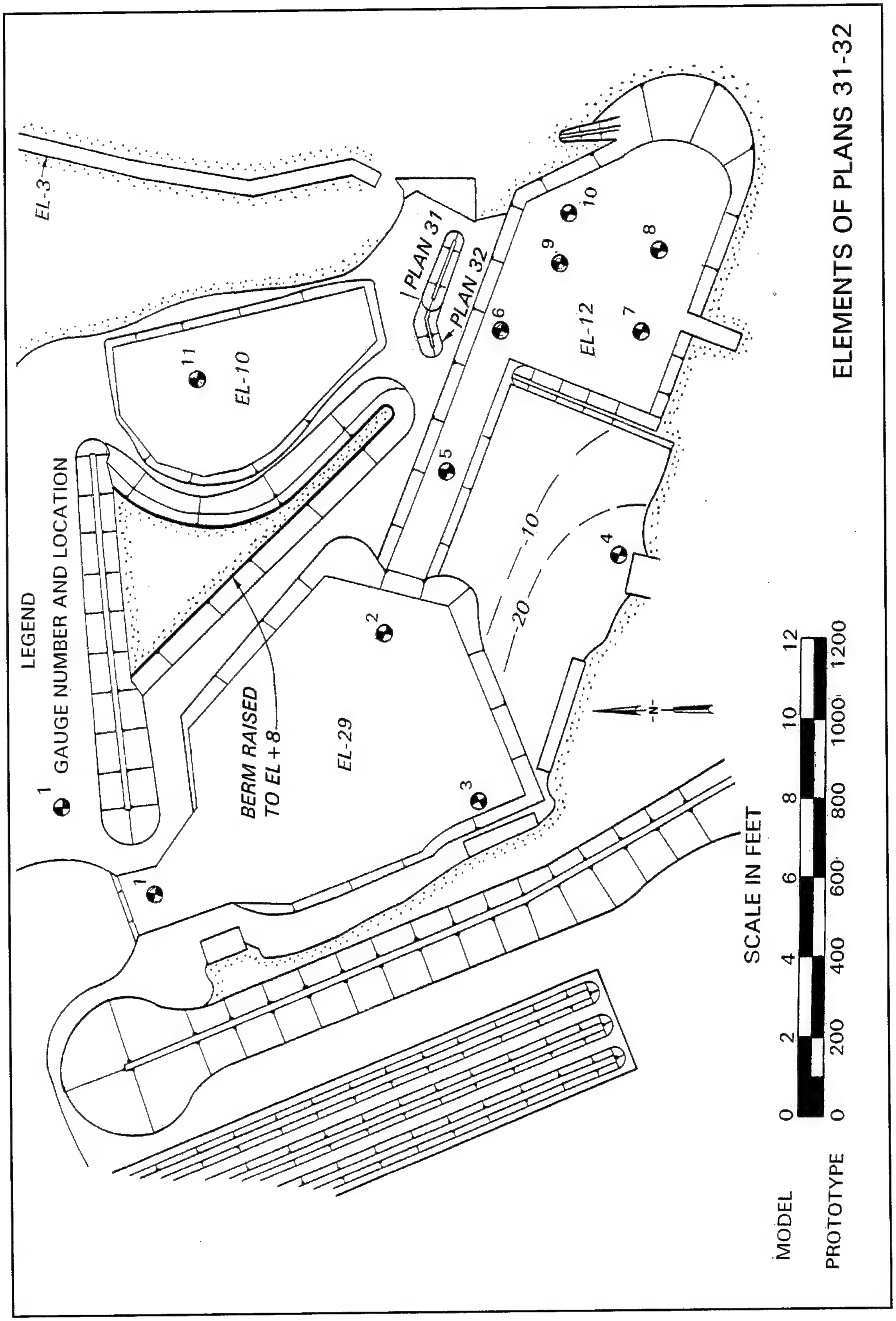


ELEMENTS OF PLAN 28

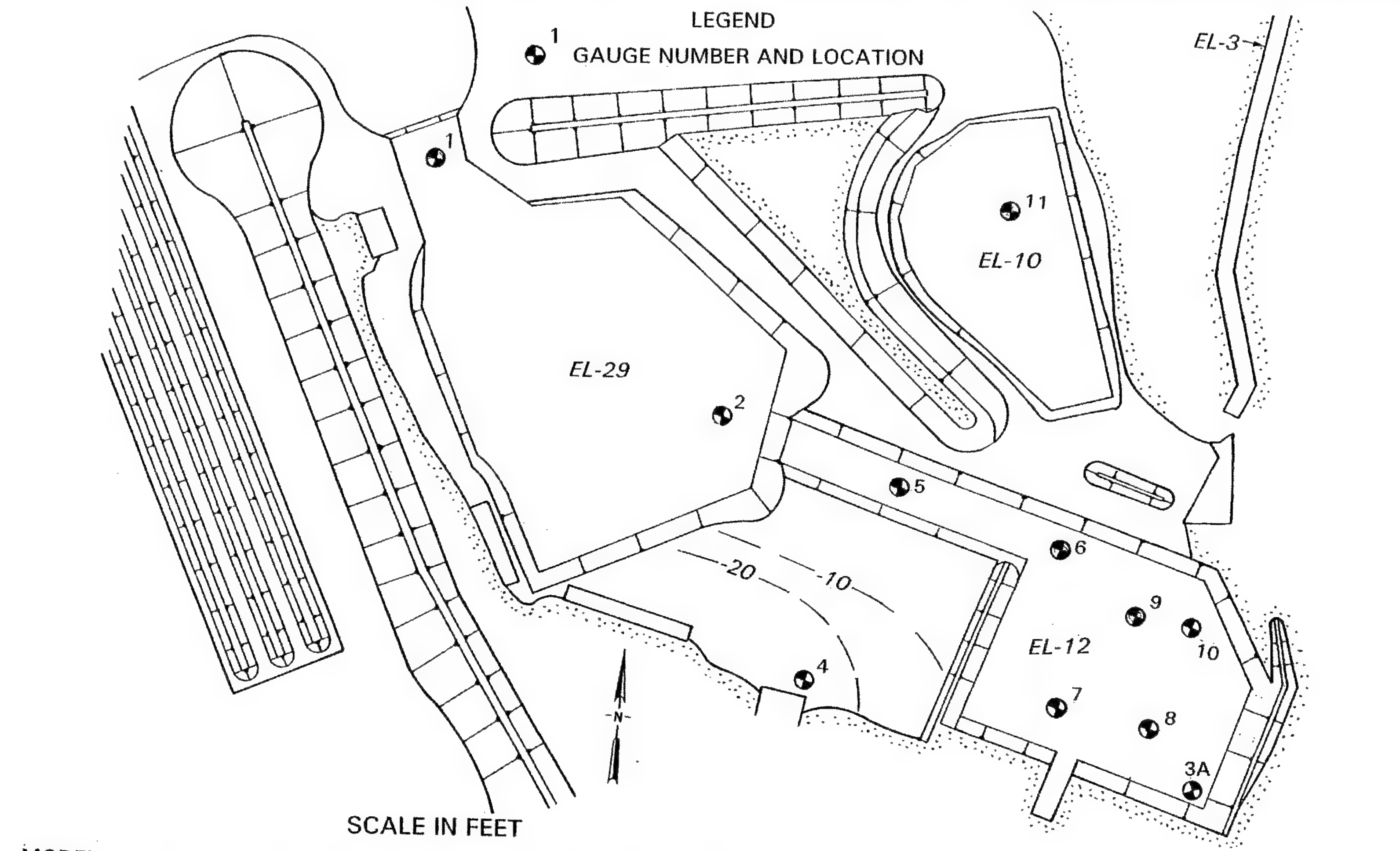


ELEMENTS OF PLANS 29-30

Plate 2



ELEMENTS OF PLANS 31-32



ELEMENTS OF PLAN 33

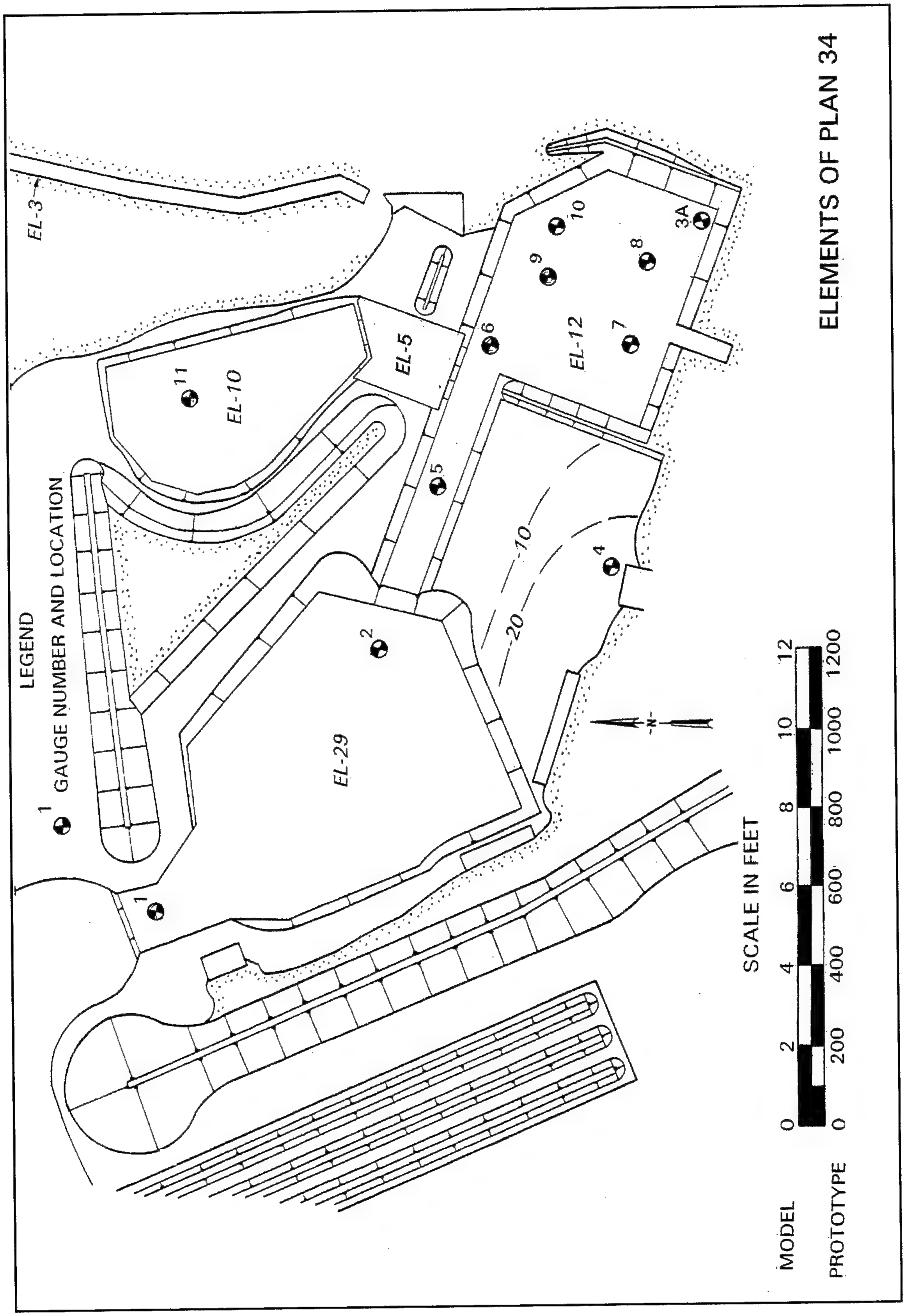

ELEMENTS OF PLAN 34
LEGEND
1 GAUGE NUMBER AND LOCATION
EL-3
EL-5
EL-10
EL-12
EL-29
1
2
3A
4
5
6
7
8
9
10
11
-10
-20
SCALE IN FEET
MODEL 0 2 4 6 8 10 12
PROTOTYPE 0 200 400 600 800 1000 1200
N

Plate 5

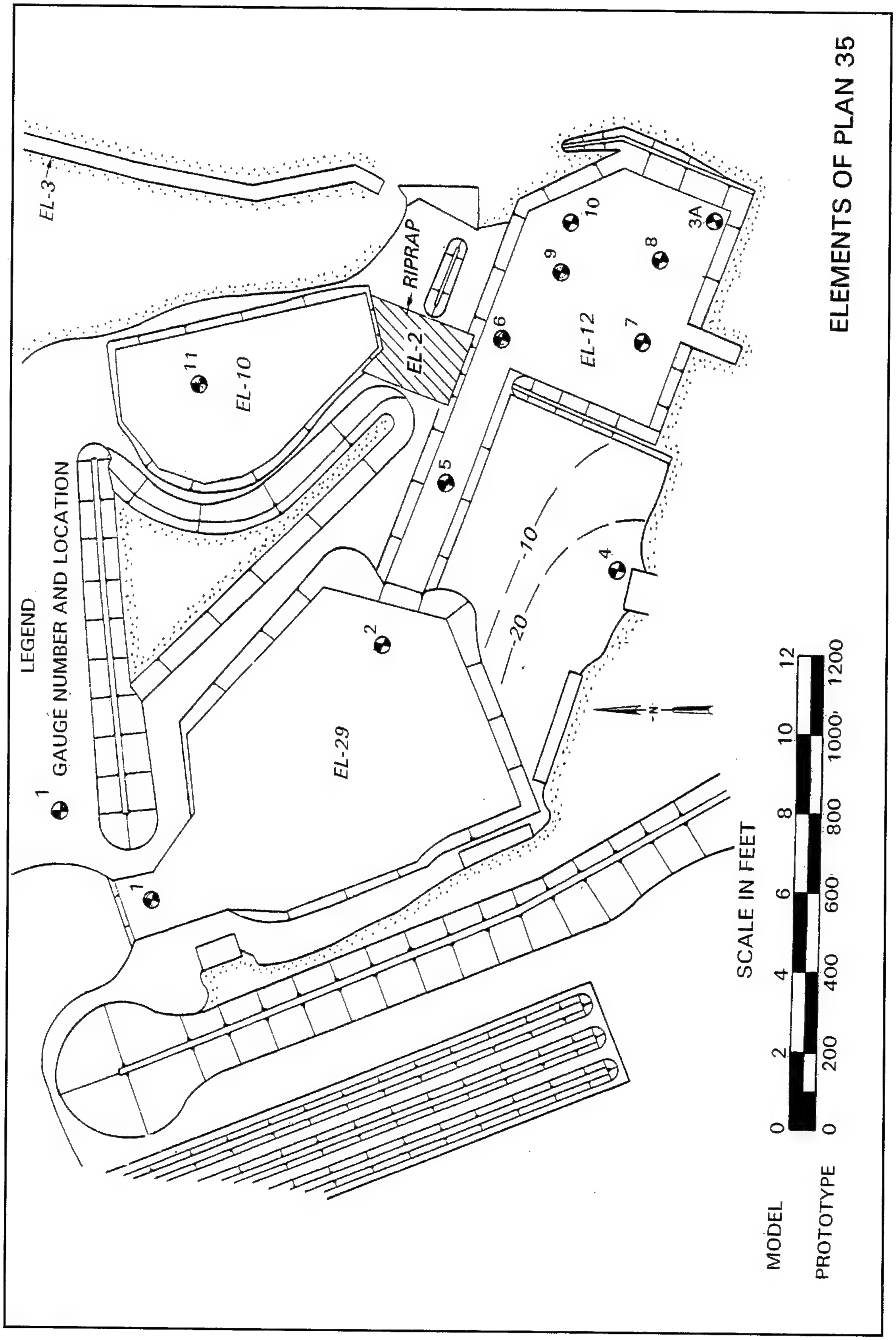
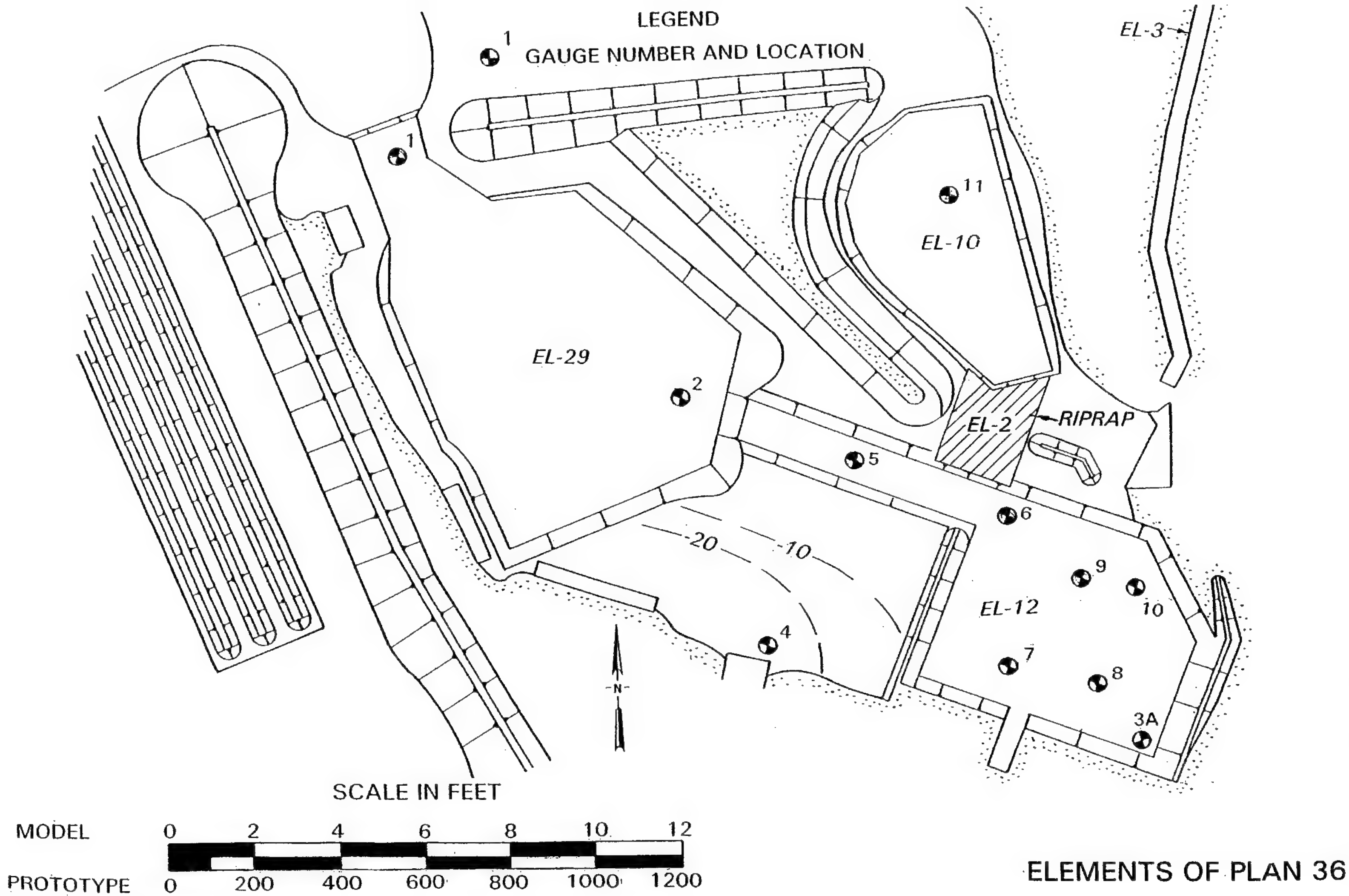
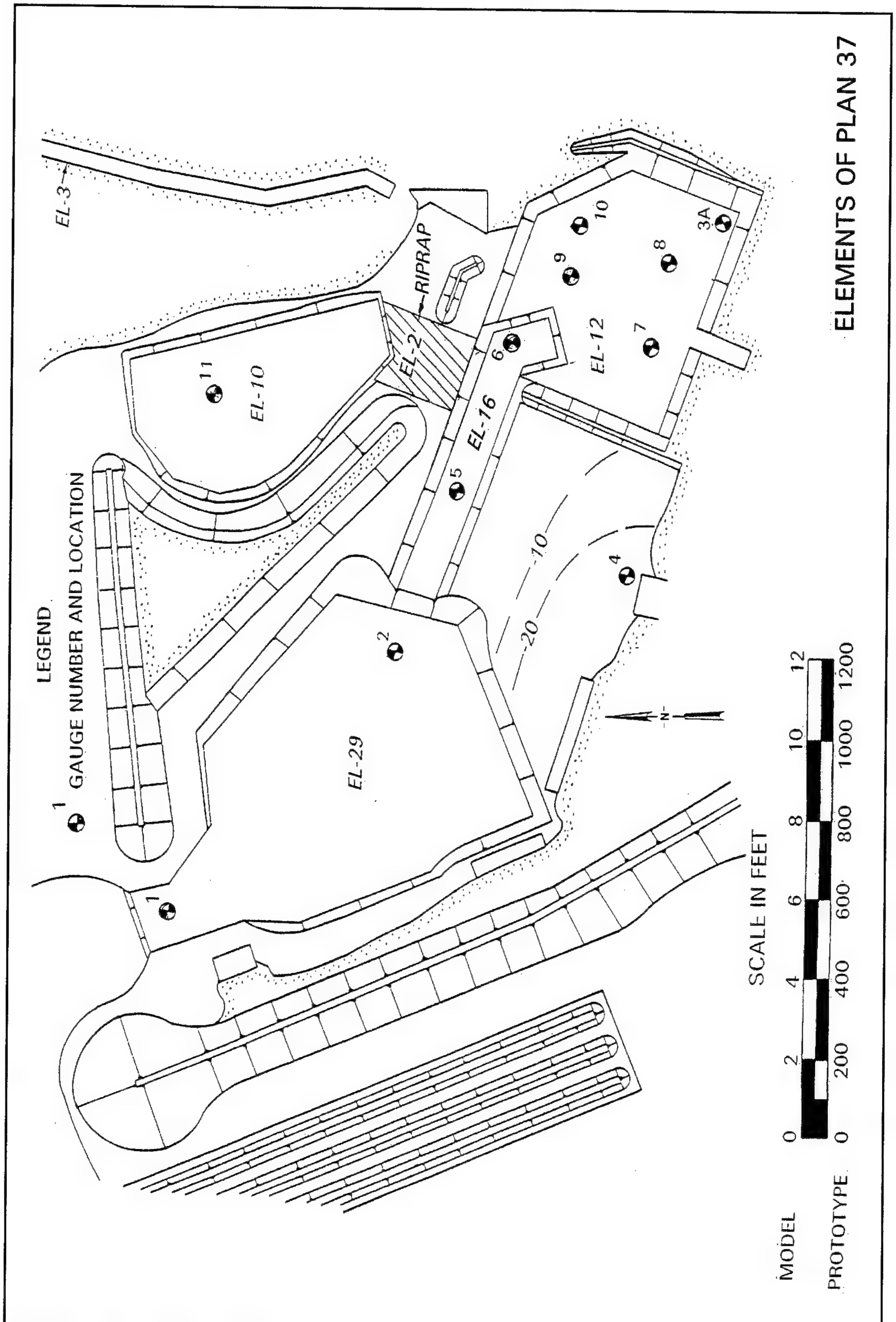

ELEMENTS OF PLAN 35
LEGEND
GAUGE NUMBER AND LOCATION
EL-3
RIPRAP
EL-2
EL-10
EL-12
EL-29
-10
-20
N
SCALE IN FEET
MODEL 0 2 4 6 8 10 12
PROTOTYPE 0 200 400 600 800 1000 1200

Plate 6

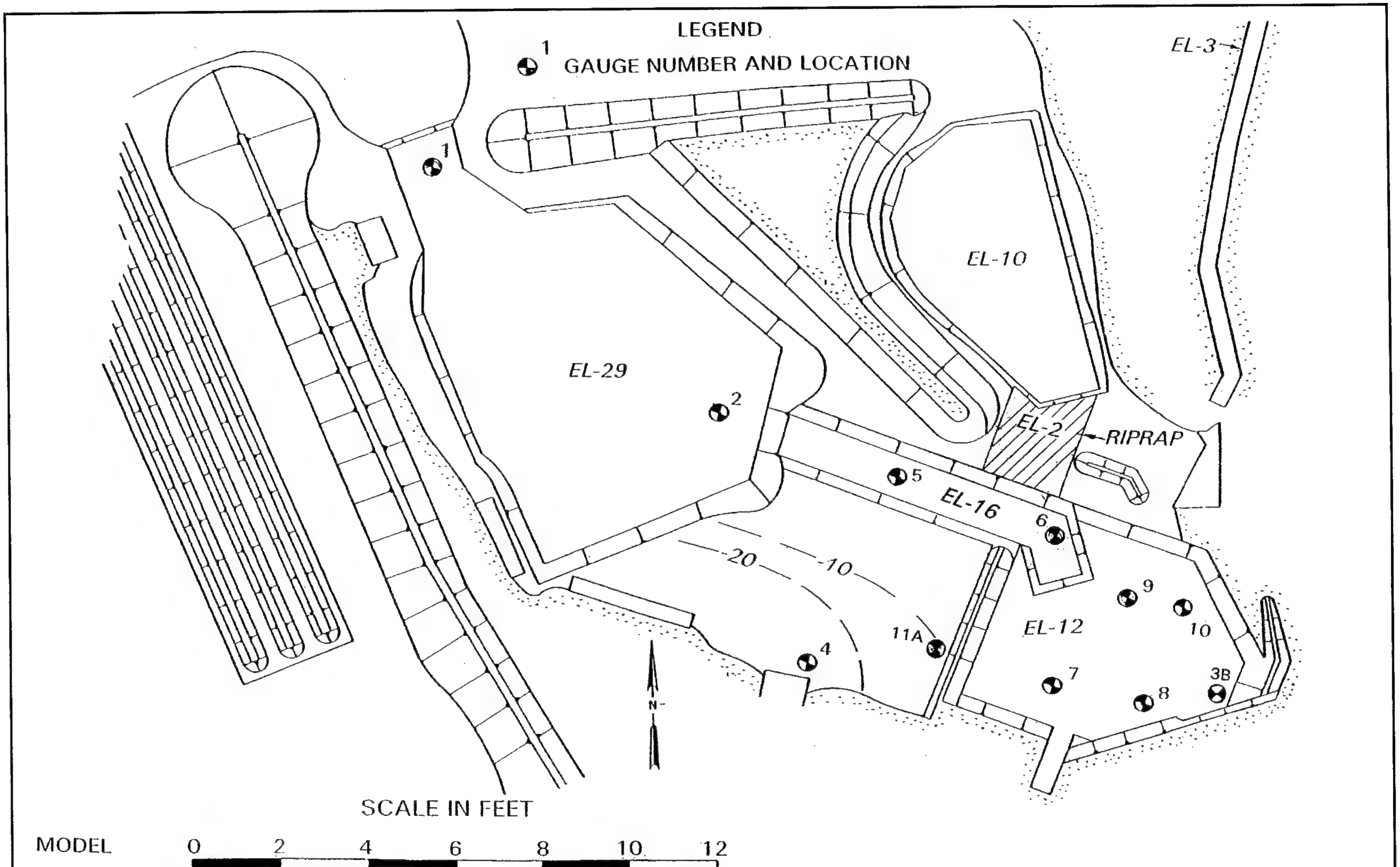


ELEMENTS OF PLAN 36

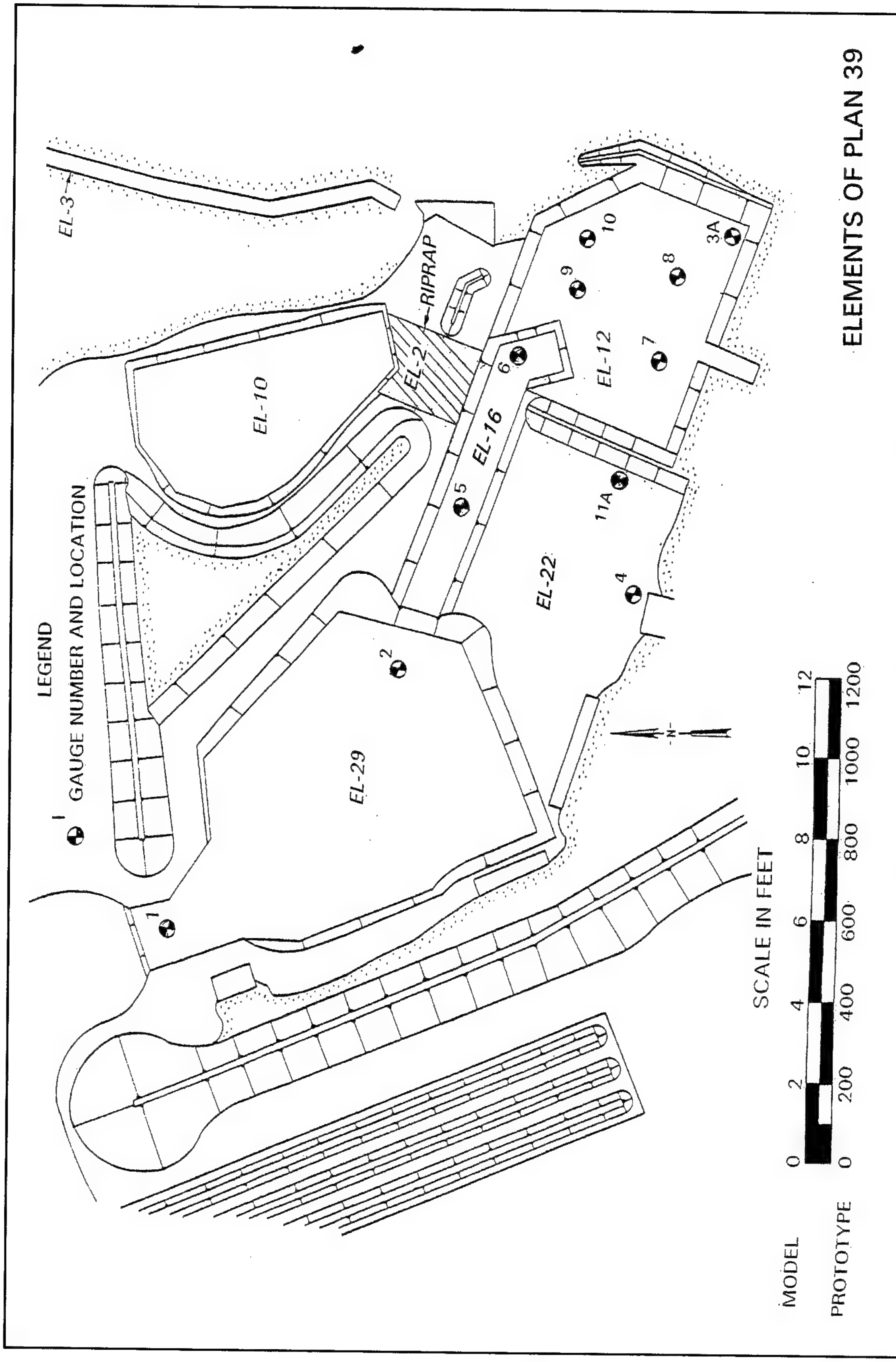


ELEMENTS OF PLAN 37

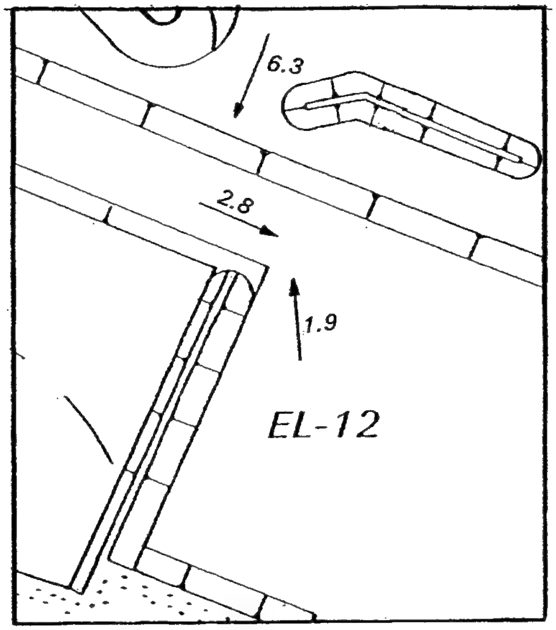
Plate 8



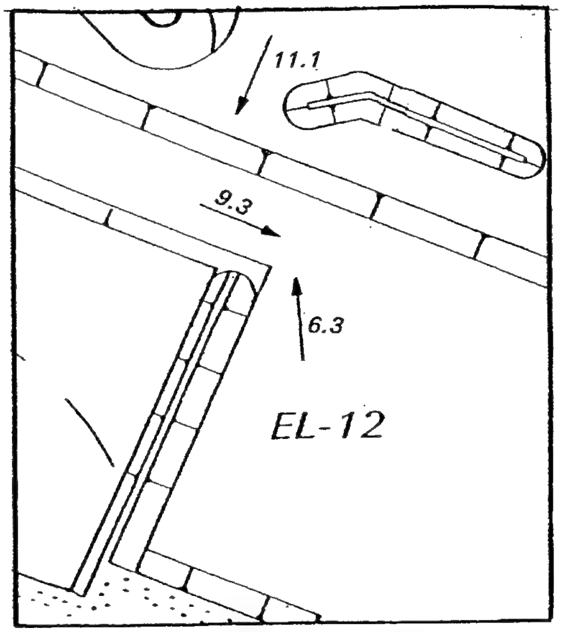
ELEMENTS OF PLAN 38



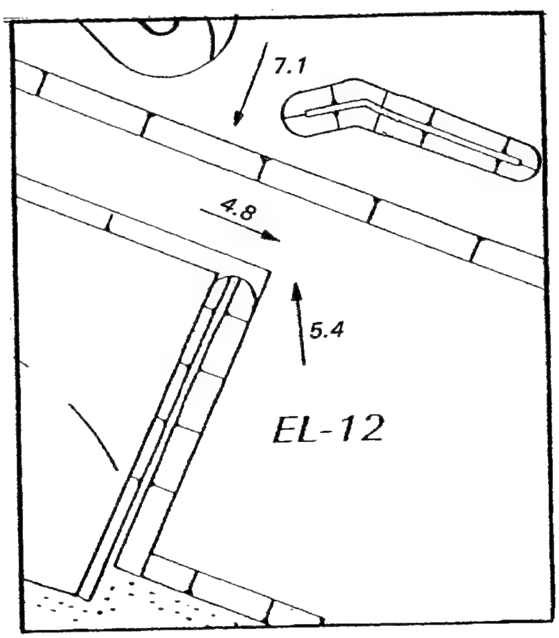
ELEMENTS OF PLAN 39



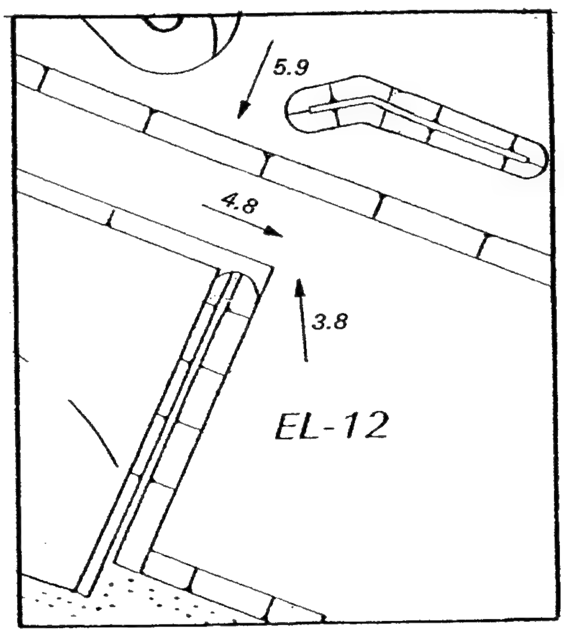
10-sec, 10-ft waves



16-sec, 19-ft waves



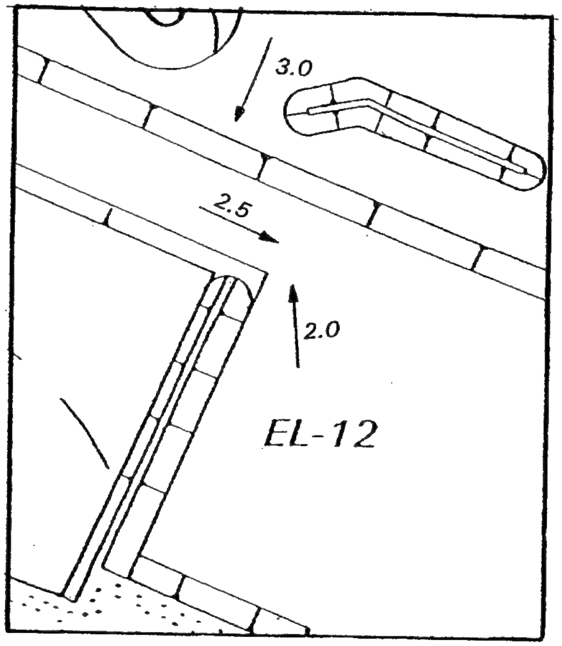
20-sec, 14-ft waves



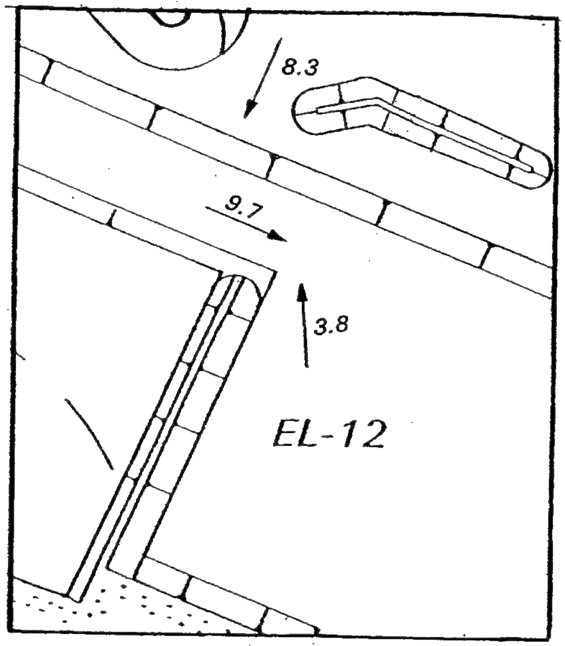
25-sec, 10-ft waves

Wave-induced current patterns and magnitudes (prototype feet per second) for Plan 32, swl= + 3.2 ft

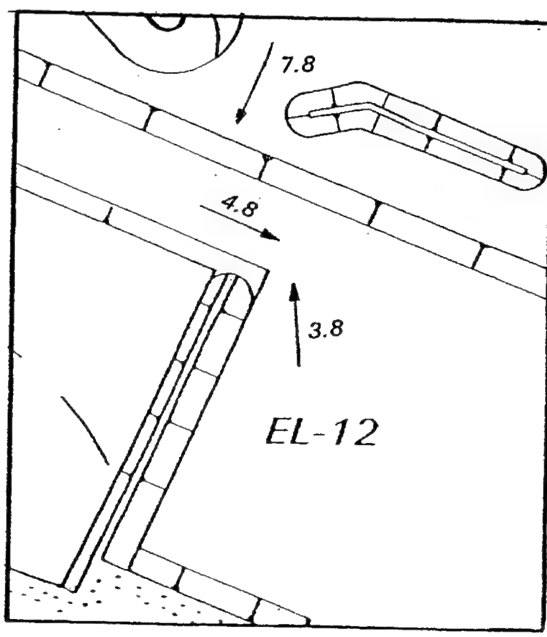
(TO CONVERT TO METERS, MULTIPLY BY 0.3048)



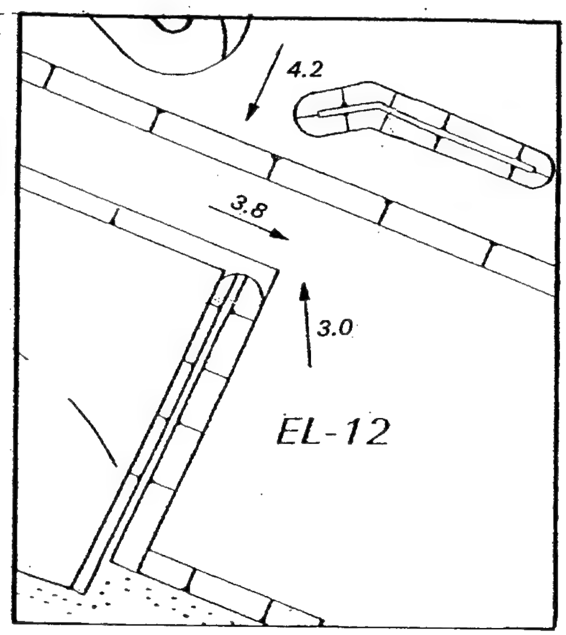
10-sec, 10-ft waves



16-sec, 19-ft waves

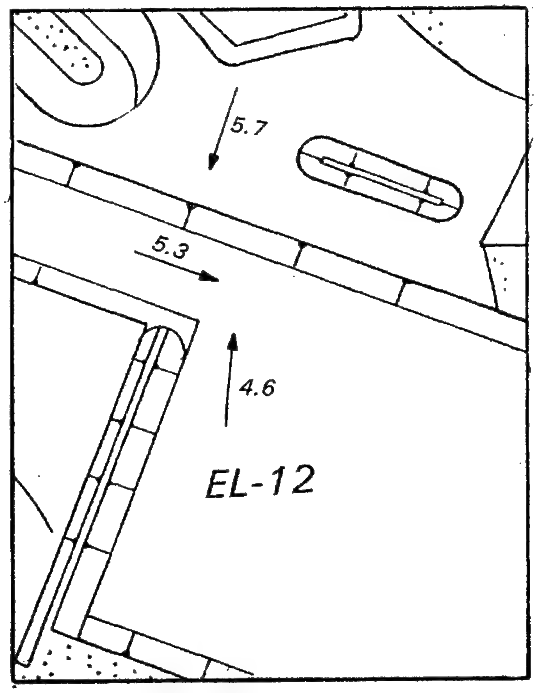


20-sec, 14-ft waves

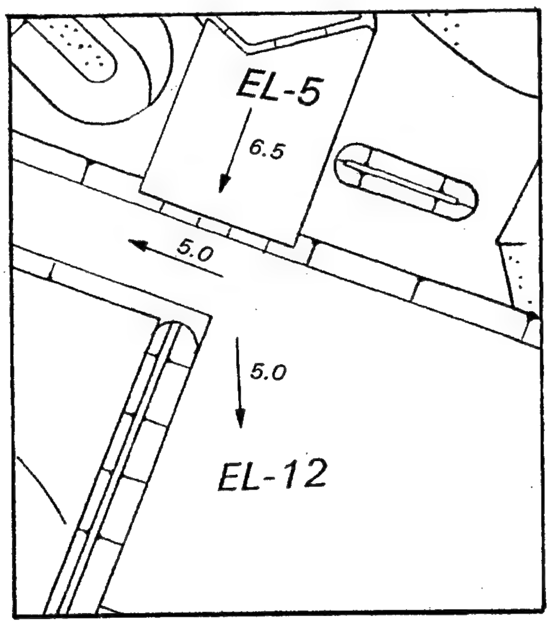


25-sec, 10-ft waves

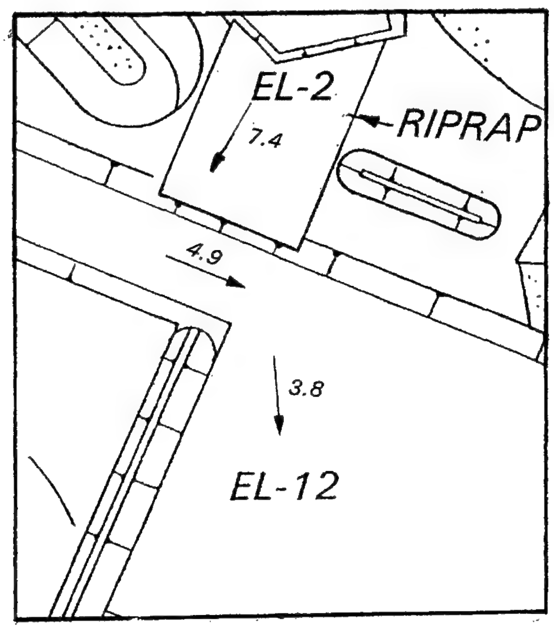
Wave-induced current patterns and magnitudes (prototype feet per second) for Plan 32, swl = +7.0 ft



Plan 33

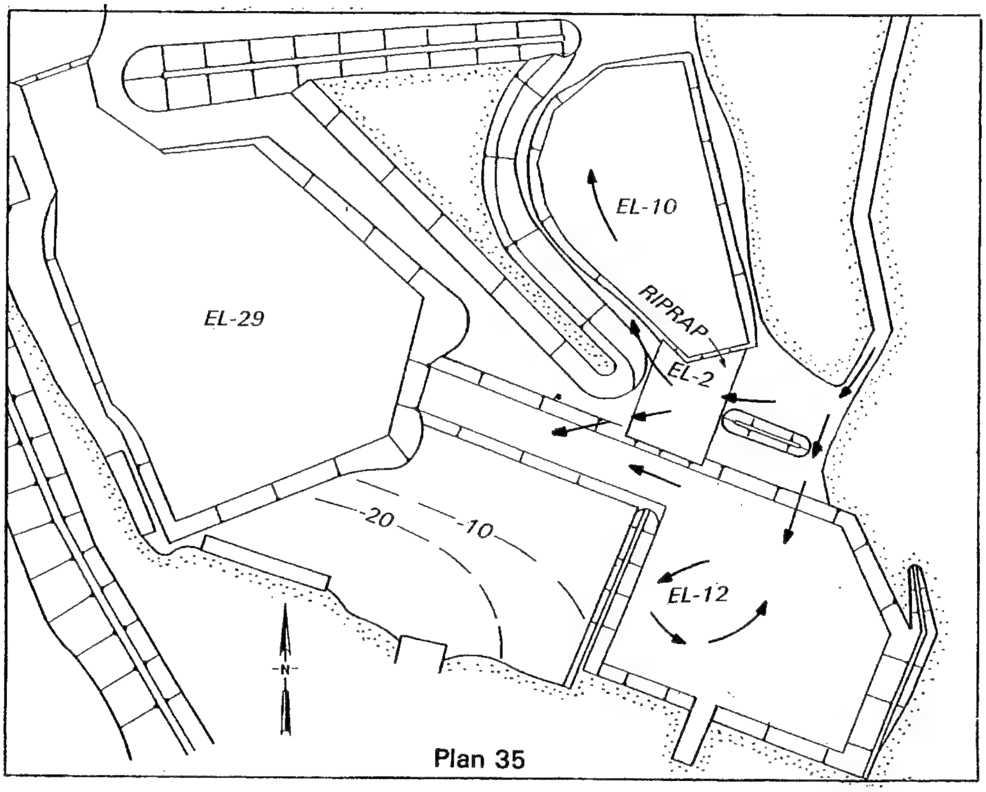


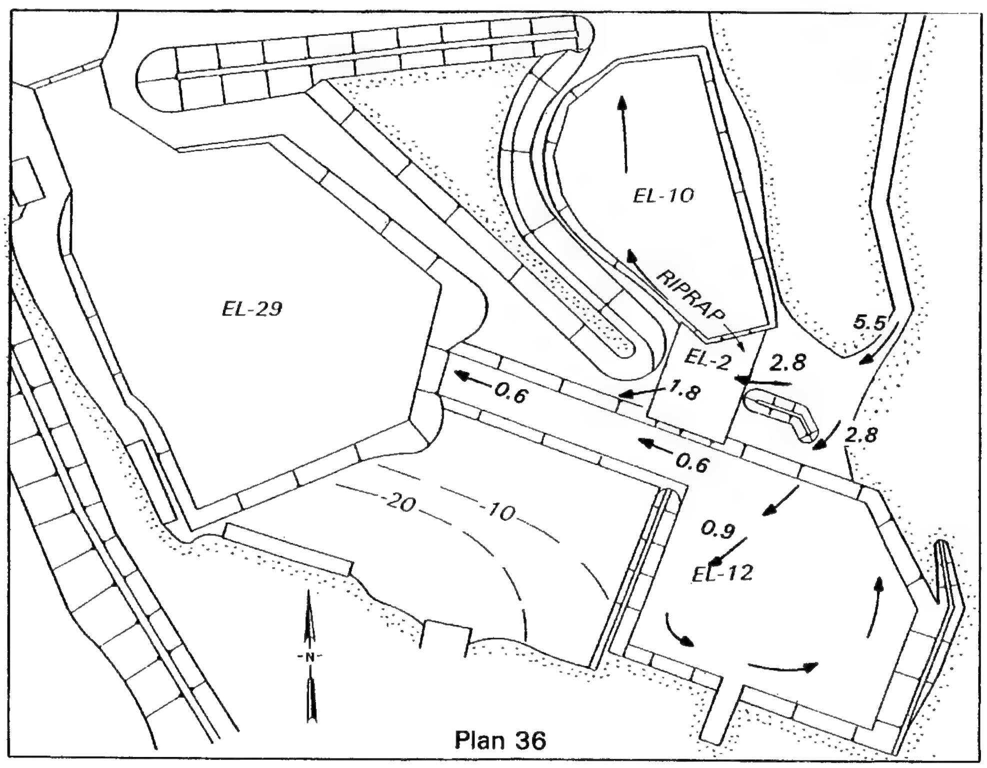
Plan 34



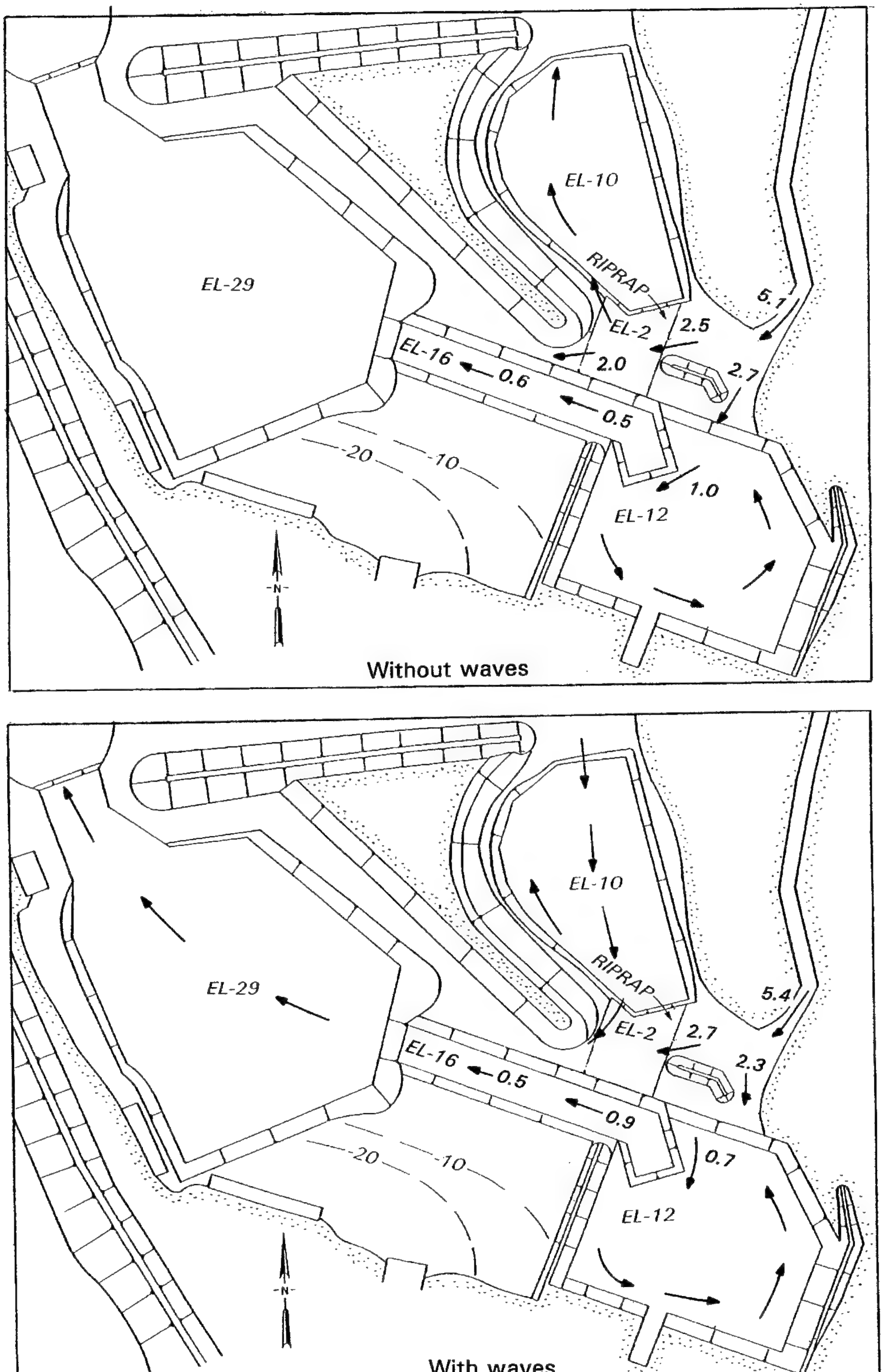
Plan 35

Wave-induced current patterns and magnitudes (prototype feet per second) obtained for Plans 33-35, 16-sec, 19-ft waves with the +3.2 ft swl

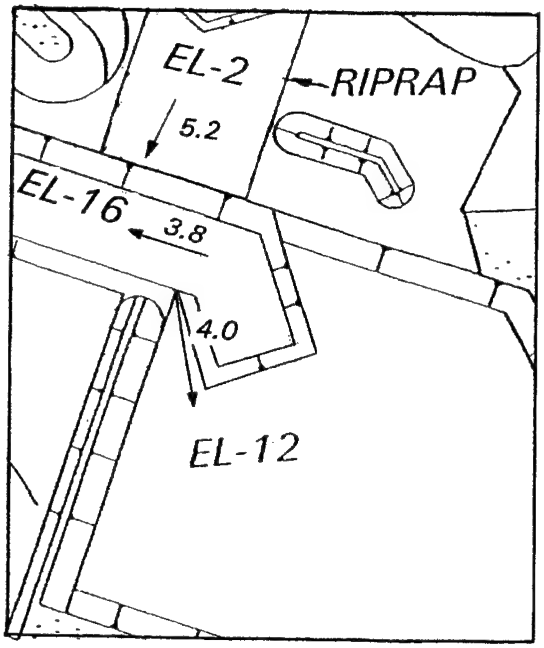




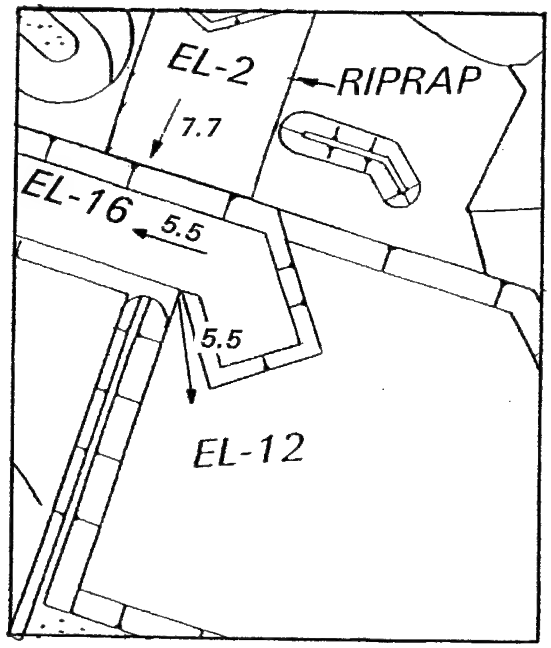
Ebb tidal currents for Plans 35 and 36 for the +7.0-ft tidal range



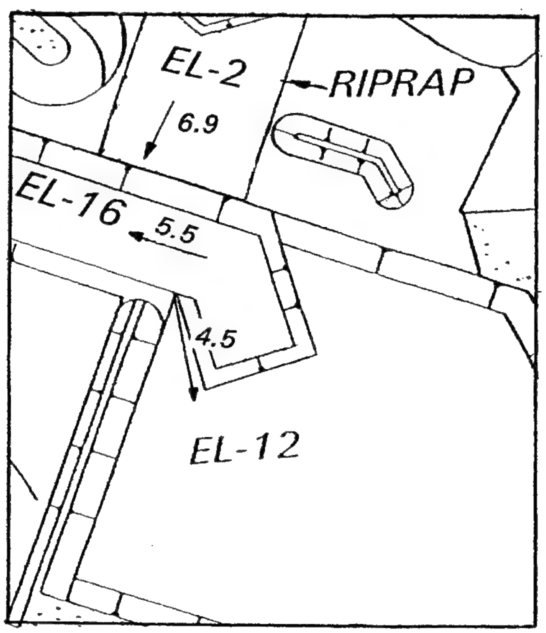
Ebb tidal currents for Plan 37 with and without waves for the +7.0-ft tidal range



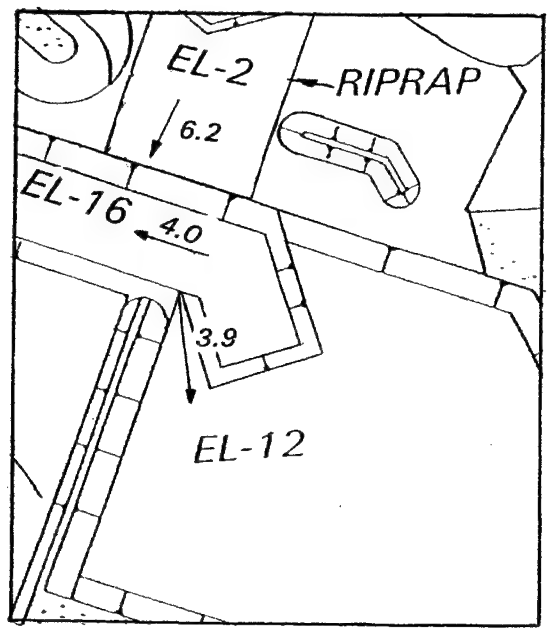
10-sec, 10-ft waves



16-sec, 19-ft waves

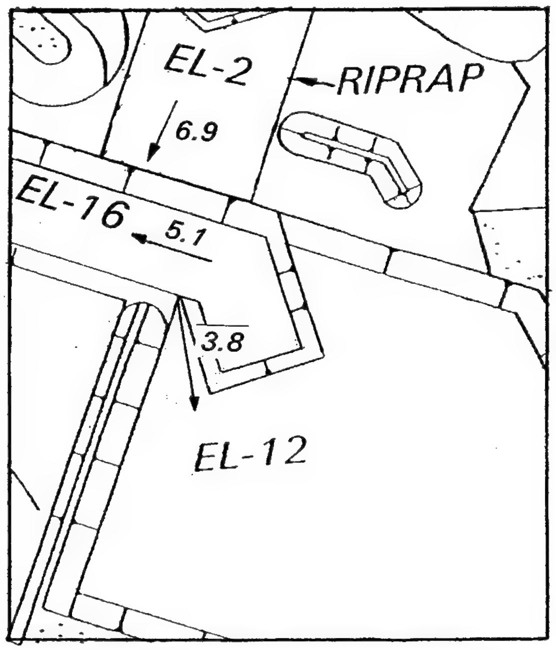


20-sec, 14-ft waves

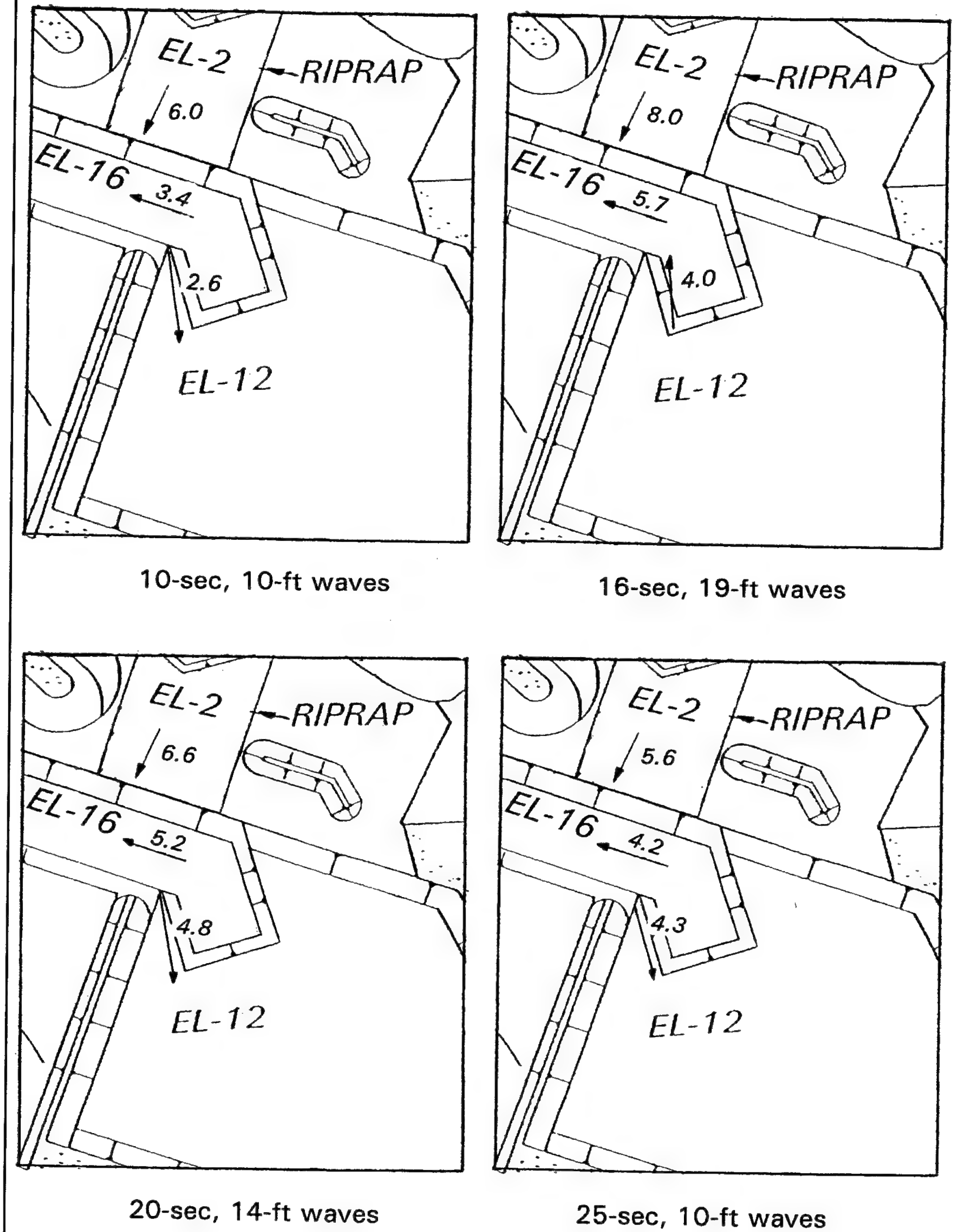


25-sec, 10-ft waves

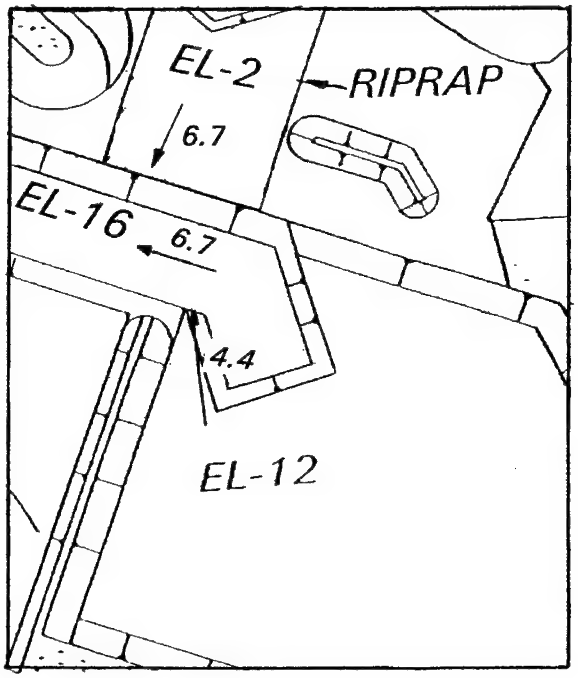
Wave-induced current patterns and magnitudes (prototype feet per second) for Plan 37, swl = +3.2 ft



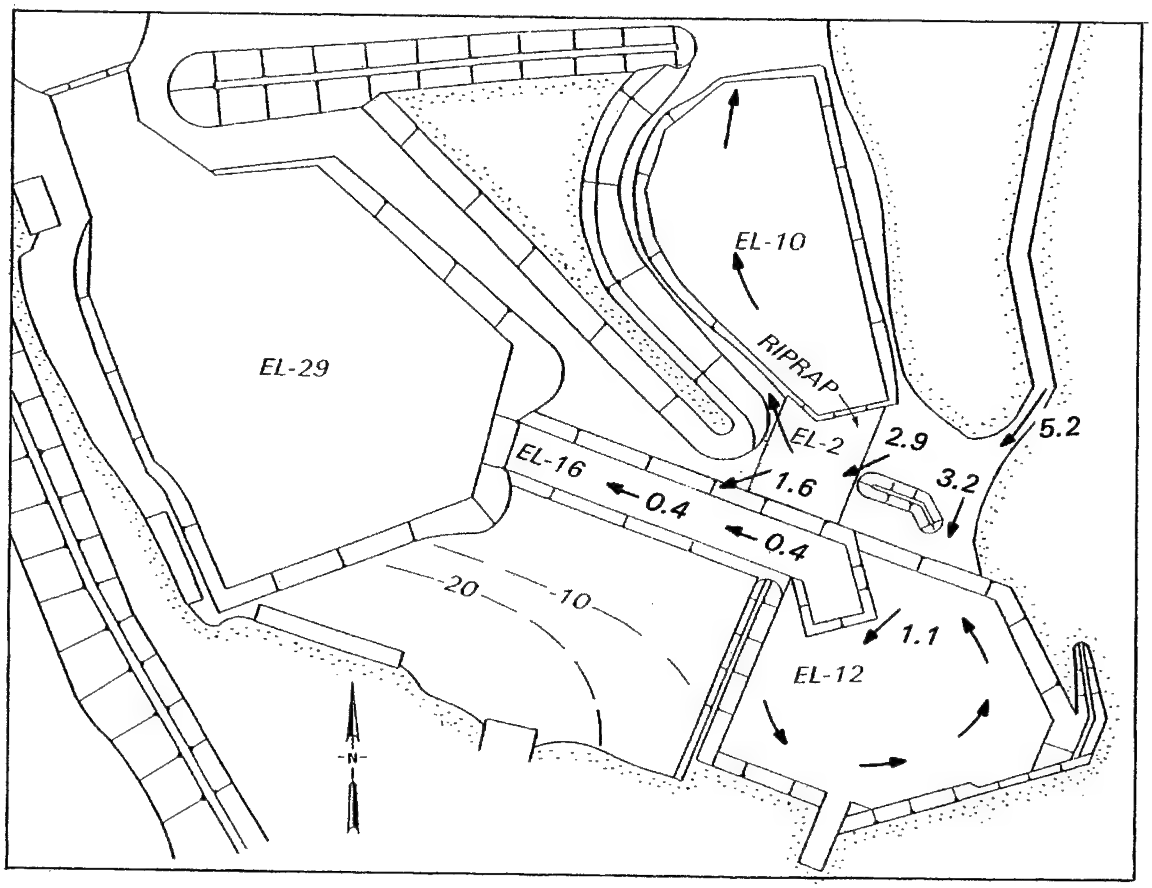
Wave-induced current patterns and magnitudes (prototype feet per second) for Plan 37, 16-sec, 19-ft waves, swl = +7.0 ft



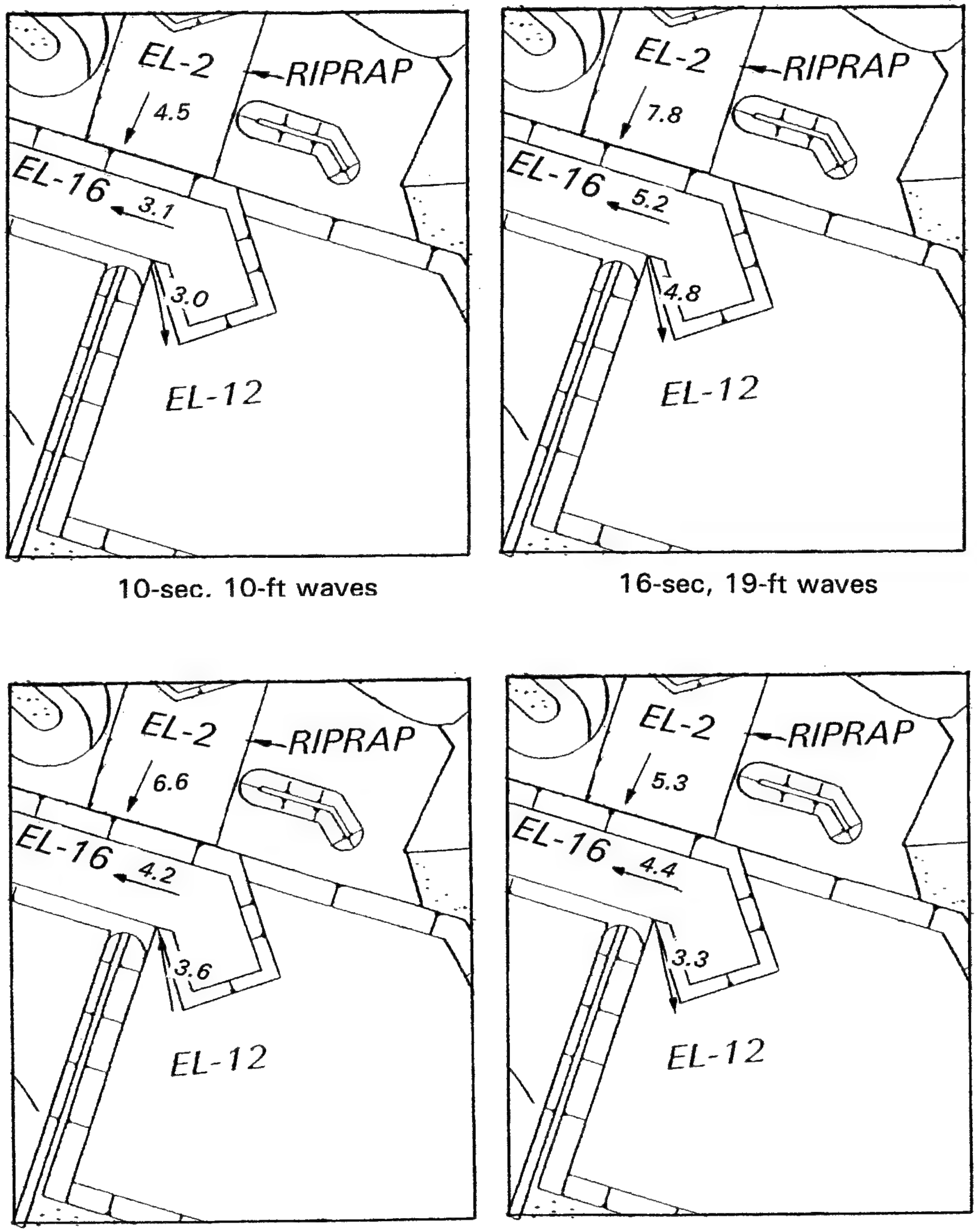
Wave-induced current patterns and magnitudes (prototype feet per second) for Plan 38, swl = +3.2 ft



Wave-induced current patterns and magnitudes (prototype feet per second) for Plan 38, 16-sec, 19-ft waves, swl = +7.0 ft



Ebb tidal currents for Plan 38 for the +7.0-ft tidal range



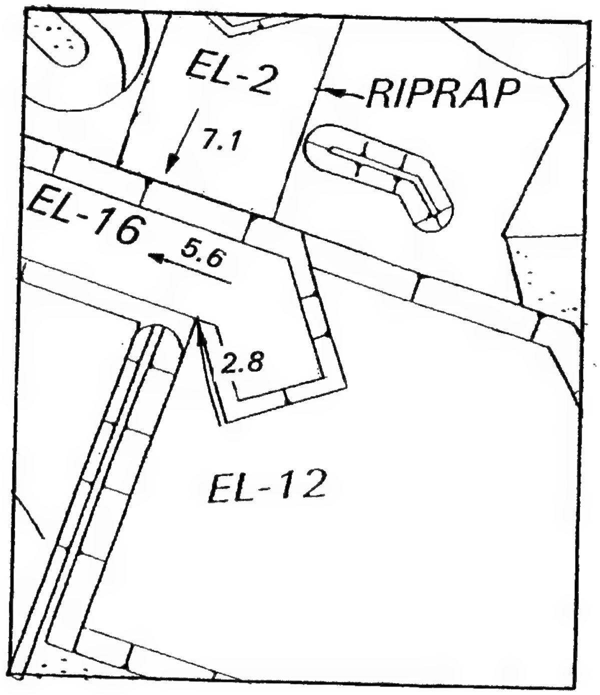
10-sec. 10-ft waves

16-sec, 19-ft waves

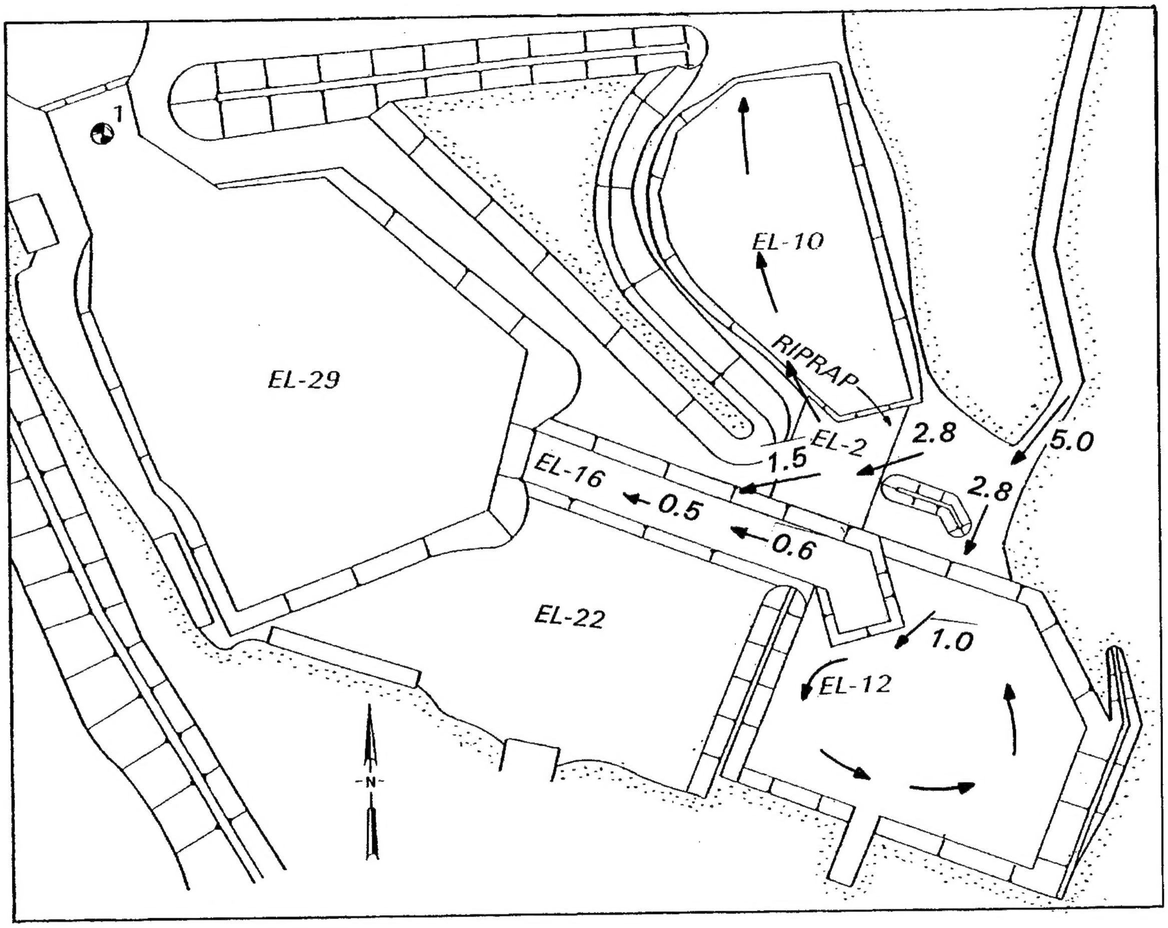
20-sec, 14-ft waves

25-sec, 10-ft waves

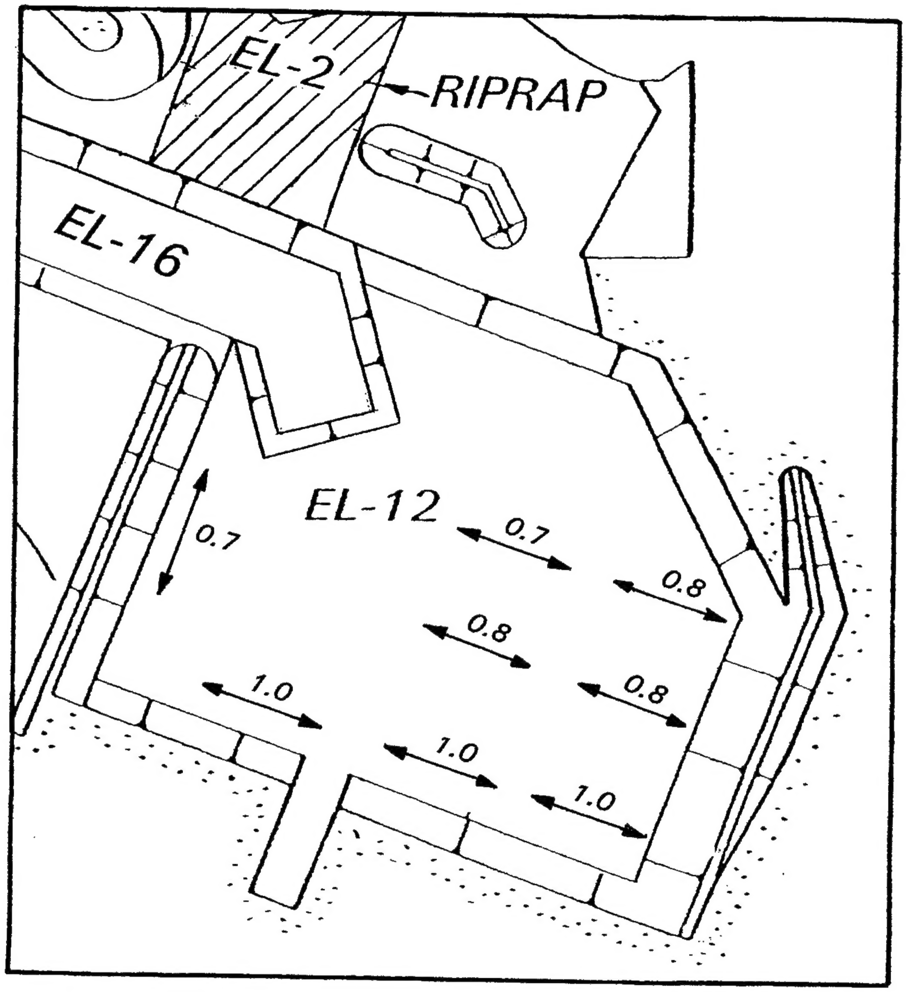
Wave-induced current patterns and magnitudes (prototype feet per second) for Plan 39, swl = +3.2 ft



Wave-induced current patterns and magnitudes (prototype feet per second) for Plan 39, 16-sec, 19-ft waves, swl = +7.0 ft



Ebb tidal currents for Plan 39 for the +7.0-ft tidal range



Current directions and maximum velocities (prototype feet per second) associated with harbor seiche for 60-vessel configuration

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. REPORT DATE *(DD-MM-YYYY)*
May 2001

2. REPORT TYPE
Final Report

3. DATES COVERED *(From - To)*

4. TITLE AND SUBTITLE
Design for Small-Boat Harbor Improvements and Tidal Flushing at St. Paul Harbor, St. Paul Island, Alaska
Coastal Model Investigation

5a. CONTRACT NUMBER

5b. GRANT NUMBER

5c. PROGRAM ELEMENT NUMBER

5d. PROJECT NUMBER

5e. TASK NUMBER

5f. WORK UNIT NUMBER

6. AUTHOR(S)

Robert R. Bottin, Jr., Hugh F. Acuff

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)

U.S. Army Engineer Research and Development Center
Coastal and Hydraulics Laboratory
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

8. PERFORMING ORGANIZATION REPORT NUMBER

ERDC/CHL TR-01-10

9. SPONSORING / MONITORING AGENCY NAME(S) AND ADDRESS(ES)

U.S. Army Engineer District, Alaska
P.O. Box 898
Anchorage, AK 99506-0898

10. SPONSOR/MONITOR'S ACRONYM(S)

11. SPONSOR/MONITOR'S REPORT NUMBER(S)

12. DISTRIBUTION / AVAILABILITY STATEMENT

Approved for public release; distribution is unlimited.

13. SUPPLEMENTARY NOTES

14. ABSTRACT

A 1:100-scale (undistorted) three-dimensional coastal hydraulic model was initially used to investigate the design of proposed harbor improvements at St. Paul Harbor, St. Paul Island, Alaska, with respect to wave and current conditions in the harbor and sediment patterns at the site. Wave-induced circulation and sediment patterns seaward of the main breakwater as a result of submerged reefs were investigated. Proposed improvements consisted of deepening the entrance channel, constructing a maneuvering area and installing a wave dissipating landfill inside the existing harbor, and constructing submerged reefs seaward of the main breakwater. The model was reactivated in 1997 to study, on a preliminary basis, small-boat harbor improvements and flushing of Salt Lagoon in St. Paul Harbor. In this study, the model was reactivated to finalize the design of small-boat harbor improvements and flushing at St. Paul Harbor. The model reproduced approximately 2,865 m (9,400 ft) of the St. Paul shoreline, the existing harbor, the surface area of Salt Lagoon with its connecting channel to the harbor, and sufficient offshore area in the Bering Sea to permit generation of the required test waves. An 18.3-m-long (60-ft-long) unidirectional, spectral wave generator and an automated data acquisition and control system were used in model operation. Conclusions from study results were as follows:

(Continued)

15. SUBJECT TERMS

Tidal flushing	Hydraulic models	Wave-dissipating landfill
Harbors	Wave-induced currents	
Wave protection	St. Paul Harbor, St. Paul Island, Alaska	

16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON
a. REPORT	b. ABSTRACT	c. THIS PAGE			19b. TELEPHONE NUMBER *(include area code)*
UNCLASSIFIED		UNCLASSIFIED		72	

14. (Concluded)

a. Preliminary experiments indicated that all improvement plans would result in wave heights of less than 0.3 m (1.0-ft) in the small-boat mooring area for short-period storm wave conditions.

b. Preliminary experiments indicated that the harbor would experience long-period (surge) conditions for all the improvement plans.

c. Preliminary experiments indicated that the area between the wave-dissipating spending beach and the interior detached breakwater should be constructed to an el of -0.6 m (-2.0 ft) to reduce wave heights in the small-boat harbor mooring areas. Excessive wave-induced currents in this area, however, indicated that the area should be hardened (capped with riprap) to prevent scour.

d. Preliminary experiments indicated that strong wave-induced currents in the interior channel may cause navigation difficulities during extreme storm wave events. Strong wave-induced currents along the area east of the shore-connected breakwater also may pose problems for vessels mooring in this vicinity. These current magnitudes also indicate that toe protection at the head of the structure may be required.

e. Preliminary experiments indicated that the angled interior detached breakwater would result in enhanced circulation and better distribution of flow in the small-boat harbor basin for ebb tidal currents as opposed to the straight structure.

f. Preliminary experiments indicated that the -4.9-m-deep (-16-ft-deep) interior channel would result in enhanced wave-induced circulation and stronger eddies in the small-boat basin as opposed to the -3.7-m-deep (-12-ft-deep) channel.

g. Experiments indicated that the 60-vessel plan configuration (Plan 37) will provide adequate wave and surge protection to the small-boat harbor as well as adequate harbor circulation.

h. Experiments indicated that the 30-vessel plan configuration (Plan 38) will provide adequate wave and surge protection to the small-boat harbor as well as adequate harbor circulation.

i. Experiments indicated that an increase in depths in the harbor to -6.7 m (-22 ft) west of the interior shore-connected breakwater (Plan 39) will have no negative impacts on wave and surge conditions or harbor circulation in the small-boat harbor.

j. Experiments indicated that long-period surge conditions exist in the harbor during storm wave events. These conditions must be properly designed for with respect to the dock systems and vessel moorage orientation in the small-boat harbor.

k. Experiments indicated that the 0.0-m (0.0-ft) el of the wave-dissipating spending beach (with the +1.2-m (+4.0-ft) berm along its perimeter) assessed during the study will provide essentially the same level of protection from storm waves in the mooring area as the +3.7-m (+12.0-ft) el spending beach tested in earlier studies.